1958 REGIONAL MEETING—DETROIT—OCTOBER 27-29, 1958

MARCH 1958
No. 9
VOL. 29

JOURNAL
OF THE AMERICAN CONCRETE INSTITUTE

THIS MONTH

Papers and Reports.....721-772

Effects of Revibrating Concrete.....C. A. VOLLICK 721

Concrete for Sewage Works.....E. C. WENGER 733

Creep of Concrete under Variable Stress..A. D. ROSS 739

Ultimate Resisting Moment of Beams with Compression Reinforcement.....EUGENE GUILLARD 759

First Slip-Formed Apartment Building in the United States.....JOHN H. DOGGETT 767

Current Reviews.....773-774

Discussion.....775-824
Discussion contents listed on back cover

News Letter 1-34

ACI
PROCEEDINGS
VOL. 54

THERE'S A NEW "H" IN HAVANA

● A huge "H" column extends upward the entire height of Havana's newest luxury apartment building, 312 feet above sidewalk level.

It forms the "spinal column" of the slender 32-story structure and is carefully designed to withstand wind stresses due to hurricanes.

The "H" is built of reinforced concrete with a compressive strength of 5000 psi. Measuring 55 feet deep, with 60-ft. flanges and a variable thickness of web, it is constructed to resist wind pressures of 120 mph at lower levels to 180 mph on upper floors.

The building, known as "17 y 0"—its street location—towers above historic Malecón Drive in the famous Vedado section. It exemplifies the dramatic beauty of the many new modern buildings designed in concrete along Havana's growing skyline.

A total of 11,000 bbls., of El Morro Cement, produced by La Compañia Cubana de Cemento Portland, Lone Star subsidiary, were used in the project.

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MARCH 1958

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PROCEEDINGS VOL. 54

JOURNAL
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Published by the American Concrete Institute under the authority of its Board of Direction

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Bound volumes 1 to 53 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1957) are for sale as far as available at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

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Remarks on ACI progress by retiring Institute president WALTER H. PRICE will head the list of April JOURNAL offerings.

•

Test results of an experimental study to determine "Strength of Concrete Under Combined Tensile and Compressive Stresses" will be presented by DOUGLAS McHENRY and JOSEPH KARNI.

•

FRANK W. CHAPPELL describes the method used to overcome severe architectural restrictions in his paper, "High Strength Steel and Concrete Result in Minimum Column Sizes."

•

Experimental techniques are described and data are presented to evaluate stresses due to nonuniform heating in "Some Physical Properties of Concrete at High Temperature" by ROBERT PHILLEO.

•

"Flexural Cracks in Reinforced Concrete Beams" by M. CHI and ARTHUR F. KIRSTEIN introduces a new concept into the problem accompanied by simplified semi-empirical equations for the determination of the average minimum spacing and the average width of cracks in the concrete.

•

P. G. FLUCK and G. W. WASHA present a general discussion of their findings on creep behavior and influencing factors in their paper "Creep of Plain and Reinforced Concrete."

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Title No. 54-39

Effects of Revibrating Concrete*

By C. A. VOLLICK†

SYNOPSIS

Tests were made to determine the effect of revibration at intervals of 1 to 4 hr after placing on properties of hardened concrete. Results show effect of revibration on compressive strength of concretes designed with varying cement contents and different admixtures. Effect on bleeding and hardening time is also given.

INTRODUCTION

Vibration of concrete has been the subject of many papers and individual research reports. Most writers agree that vibration has been one of the most important developments in placing high quality concrete. Much less is known about revibration, and many engineers have expressed the opinion that partially set concrete should not be disturbed.

The first reported experience with revibration occurred during World War II. The American Bureau of Shipping supervised the design and construction of several concrete tankers and cargo ships, and revibration is credited with having removed water entrapped beneath longitudinal reinforcement, and increasing bond of concrete to reinforcing steel. It is also reported that revibration was used in the construction of New York City's Pier 57 and the Tappan Zee Bridge. A set retarder was used in the concrete in these structures so that each lift of concrete could be combined with the underlying lift and form a homogeneous structure without cold joints.[1]

This paper reports on tests made to determine some of the effects of revibration at predetermined intervals after placing on the properties and appearance of plastic and hardened concrete. Ready-mixed concrete was used in one set of tests. Two series of mixes were made in the laboratory, one containing $4\frac{1}{2}$ sacks, the other containing $5\frac{1}{2}$ sacks of cement per cu yd. Mixes in each series of tests included plain concrete, air-entrained concrete, set-retarded concrete, and concrete containing both an air-entraining agent and a retarder.

MATERIALS AND CONCRETE MIXES

Ready-mixed concrete

Concrete obtained from a local ready-mixed concrete plant was placed in wood boxes measuring 1 x 1 x 2 ft and vibrated. The period of vibration was varied from 6 to 60 sec; from

*Received by the Institute Mar. 7, 1957. Title No. 54-39 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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observations of the effect on the concrete, it was decided that 40 sec was sufficient to consolidate the concrete.

Four mixes of 1 cu yd each, were obtained from the ready-mixed concrete plant. All contained natural sand, New Jersey crushed traprock graded to 1-in. maximum size, and approximately 6 sacks of Type I cement per cu yd. The plant obtained aggregates from more than one source, and materials used in the mixes were not identical.

Water was adjusted at the site to provide 3-in. slump. Three of the mixes contained admixtures which were added to a portion of the mixing water. Mix 1 did not contain an admixture; a set retarder was added to Mix 2; Mix 3 contained an air-entraining agent and a set retarder; and Mix 4 contained an air-entraining agent. The concrete plant added hot water to the air-entrained mix and the water required for 3-in slump was therefore above the amount normally required. The air-entraining agent used was a neutralized Vinsol resin solution and the set retarder was of the hydroxi-adipic acid type in solution form. Table 1 summarizes properties of the fresh concrete.

The concrete was mixed 5 min after all water and admixtures were added. Approximately one wheelbarrow load, containing the first concrete discharged from the mixer, was discarded. Slump and air content were then determined and a portion of the concrete was screened through a No. 4 sieve for rate-of-hardening tests. The remaining concrete was placed in 2 ft high boxes which had dividers spaced at 1-ft intervals. Concrete blocks obtained measured 1 x 1 x 2 ft.

Laboratory mixes

Two series of mixes were made in the laboratory, one series containing 4½ sacks cement per cu yd, and one containing 5½ sacks cement per cu yd. Type I cement, crushed traprock graded to 1-in. maximum size, and natural sand were used in all mixes. Aggregate grading was as follows:

Rock		Sand	
Sieve size	Cumulative percent retained	Sieve size	Cumulative percent retained
1-in.	0	No. 4	1
¾-in.	24	No. 8	11
½-in.	70	No. 16	26
⅜-in.	95	No. 30	50
No. 4	100	No. 50	81
		No. 100	95
		Pan	100
		Fineness modulus	2.64

The coarse aggregate was sharp and contained little No. 4 to ⅜-in. size. Although the gradings were not ideal, the aggregates represented locally available material.

All mixes were proportioned for 3-in. slump. Each series included plain concrete, air-entrained concrete, retarded concrete, and air-entrained concrete with retarder. Vinsol resin air-entraining agent and hydroxi-adipic acid retarder were used, as in the ready-mixed concrete. Table 1 summarizes properties of these mixes.

Before each test mix was made, the mixer was charged with similar concrete and this concrete discarded. All materials were then placed in the mixer and the concrete mixed 3 min. Concrete was discharged into a mixing pan and turned over with a shovel to insure uniformity. Slump, air content, and unit weight were determined. Cylinders (6 x 12 in.) to be used for compressive strength tests were filled in one lift without rodding. Five 2-gal. metal containers were filled to a depth of 9 in. in one lift for bleeding tests.

TABLE 1—PROPERTIES OF FRESH CONCRETE USED IN THE TESTS

Kind of concrete	Laboratory-mixed								Ready-mixed			
Cement content	4½ sacks per cu yd				5½ sacks per cu yd				6 sacks per cu yd			
Admixture	None	Air	Retarder	Air and retarder	None	Air	Retarder	Air and retarder	None	Air	Retarder	Air and retarder
Slump, in.	3¾	4	3¼	3¾	3	2⅞	3⅛	2¾	3¼	2¾	3⅛	3
Air content before vibration, percent	1.2	6.1	1.0	5.0	1.1	5.0	1.5	5.1	2.0	4.6	2.6	3.8
Unit weight of fresh concrete, lb per cu ft	155.2	146.8	156.9	—	155.0	151.0	155.8	151.0	154	149.6	152.7	150.9
W/C ratio, by weight	0.75	0.70	0.71	0.61	0.62	0.51	0.58	0.50	0.63	0.63*	0.56	0.54

*Hot water

TABLE 2—COMPRESSIVE STRENGTH TEST RESULTS

	28-day compressive strength, psi, average of two 6 x 12-in. cylinders								28-day compressive strength, psi, indicated by impact hammer			
Cement content, mix type	4½ sacks per cu yd, laboratory-mixed				5½ sacks per cu yd, laboratory-mixed				6 sacks per cu yd, ready-mixed			
Admixture	None	Air	Retarder	Air and retarder	None	Air	Retarder	Air and retarder	None	Air	Retarder	Air and retarder
Vibration												
40 sec initial vibration	2690	2480	2920	3720	3480	3530	4060	4830	4000	3710	4550	4500
20 sec initial, 20 sec revibration at 1 hr	2890	2830	3120	3740	3730	3770	4170	4990	4650	3930	5200	4750
20 sec initial, 20 sec revibration at 2 hr	2650	2860	3200	3820	4130	3900	4330	5350	5000	4070	4800	4900
20 sec initial, 20 sec revibration at 3 hr	2690	2830	3420	4020	3590	4180	4170	5350	4500	3800	4880	4900
20 sec initial, 20 sec revibration at 4 hr	2920	2680	3200	4290	3620	3760	4340	5220	4200	3470	4900	4600

VIBRATION PROCEDURE AND LABORATORY TESTS

Vibration of test blocks

Concrete in the 1 x 1 x 2-ft blocks was vibrated with an electric vibrator operating at approximately 6000 rpm. The spud was first inserted 3 in. from the front and 3 in. from the right side. The second insertion was 3 in. from the back and 3 in. from the left side. The period of vibration for each insertion was 20 sec.

The first block in each series of tests was vibrated once in each location for a total of 40 sec immediately after the concrete was placed. All other blocks were vibrated once for 20 sec. After 1 hr one block was vibrated in the second location for 20 sec. Other blocks received the second vibration after 2, 3, and 4 hr delay.

The blocks were covered with canvas, kept moist to prevent rapid drying. After 7 days, the canvas and forms were removed and the blocks allowed to air dry until 28 days, at which time they were tested with the impact hammer.

Vibration of 6 x 12-in. cylinders

Laboratory-mixed concrete was placed in 6 x 12-in. cardboard cylinder molds in one lift, and vibrated with an electrically driven vibrator having a 1-in. spud. Two cylinders in each series received an initial vibration of 40 sec each with one insertion of the vibrator. All other cylinders were vibrated 20 sec with one insertion of the spud immediately after placing, and were revibrated after 1, 2, 3, and 4 hr respectively. Total vibration of each cylinder was 40 sec.

All concrete cylinders were stored in lime water after they had been removed from their molds. They were tested in compression when they were 28 days old.

Rate of hardening of concrete

The hardening time of all concrete was determined by the Proctor needle penetration test.[2] Concrete was wet-screened through a No. 4 sieve and the mortar placed in five 1-gal. metal containers. The mortar in one container was vibrated 40 sec by two insertions of the 1-in. spud vibrator. The mortar in the remaining containers was vibrated 20 sec initially with one insertion of the vibrator. After 1 hr, mortar in the second container was revibrated an additional 20 sec by one insertion of the vibrator; and mortar in the remaining containers was revibrated after 2, 3, and 4 hr respectively.

All containers were covered immediately after wet-screening and remained covered except when the concrete was vibrated or penetration tests were made. Penetration resistance was measured every 30 min until the Proctor needle penetration resistance was 4000 psi.

Bleeding of concrete

Five metal containers measuring 8 in. in diameter and 10 in. high were filled to a depth of 9 in. in one lift with concrete from each mix. Weight of the concrete in each container was determined and the quantity of mixing water present was calculated from the original mix. Concrete in one container was vibrated 40 sec with two insertions of the 1-in. spud vibrator; concrete in the remaining containers was vibrated 20 sec with one insertion of the vibrator. Immediately after vibration, the surfaces of the concrete were lightly troweled.

The containers were covered and placed on blocks so that the front of each container was approximately 2 in. lower than the back. Every 30 min until bleeding stopped, water was removed from containers with a pipette and measured. Concrete in containers which had been vibrated 20 sec initially, was revibrated an additional 20 sec at 1, 2, 3, and 4 hr respectively. After revibration, the bleeding test was continued.

TEST RESULTS

Strength measurement of concrete blocks

Lines spaced 2 in. apart were drawn across the face of each test block. When the blocks were 28 days old, ten readings were taken with the impact hammer in each 2-in. interval, representing 24 sq in. Approximately 120 readings made on the face of each block were averaged to determine the strength indicated by the hammer.

Strength results as indicated by the hammer are given in Table 2 and Fig. 1. Apparent strength was increased a maximum by revibration at 1 or 2 hr. Revibration at 3 or 4 hr showed less strength gain. Concrete made with en-

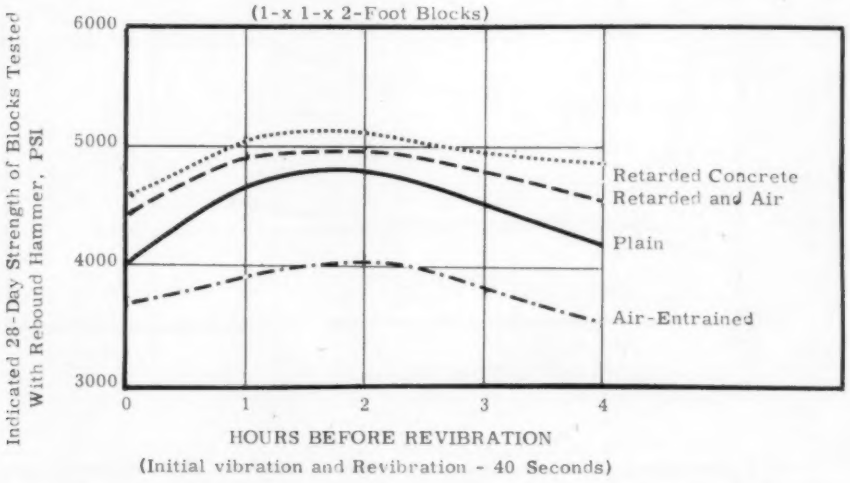

Fig. 1—Effect of revibration on strength indicated by impact hammer

trained air and revibrated at 4 hr had slightly less strength than similar concrete vibrated the same length of time initially. In all other cases revibrated concrete had higher strength. The air-entrained concrete was made with hot water and it hardened more rapidly than the other mixes which may account for the slight strength reduction. Hardening time was measured with the Proctor needle and the vibration limit suggested by Tuthill and Cordon[2] was reached in approximately 4 hr. Maximum increases in strength resulting from revibration varied from 8.9 to 22.5 percent and averaged 13.9 percent in all of the mixes.

Compressive strength of 6 x 12-in. cylinders

The concrete cylinders were removed from lime water when they were 28 days old. They were capped with a mixture containing three parts rock flour and one part sulfur, and tested for compressive strength according to standard procedure.

Results of the compressive strength tests are given in Table 2 and are plotted in Fig. 2. Revibration produced higher strengths than the same amount of initial vibration. Maximum strength increase resulting from revibration varied from 8.5 percent to 17.1 percent and averaged 14.0 percent for the mixes containing 4½ sacks of cement per cu yd. Maximum strength increase of the 5½-sack mixes varied from 6.9 percent to 18.7 percent and averaged 13.7 percent.

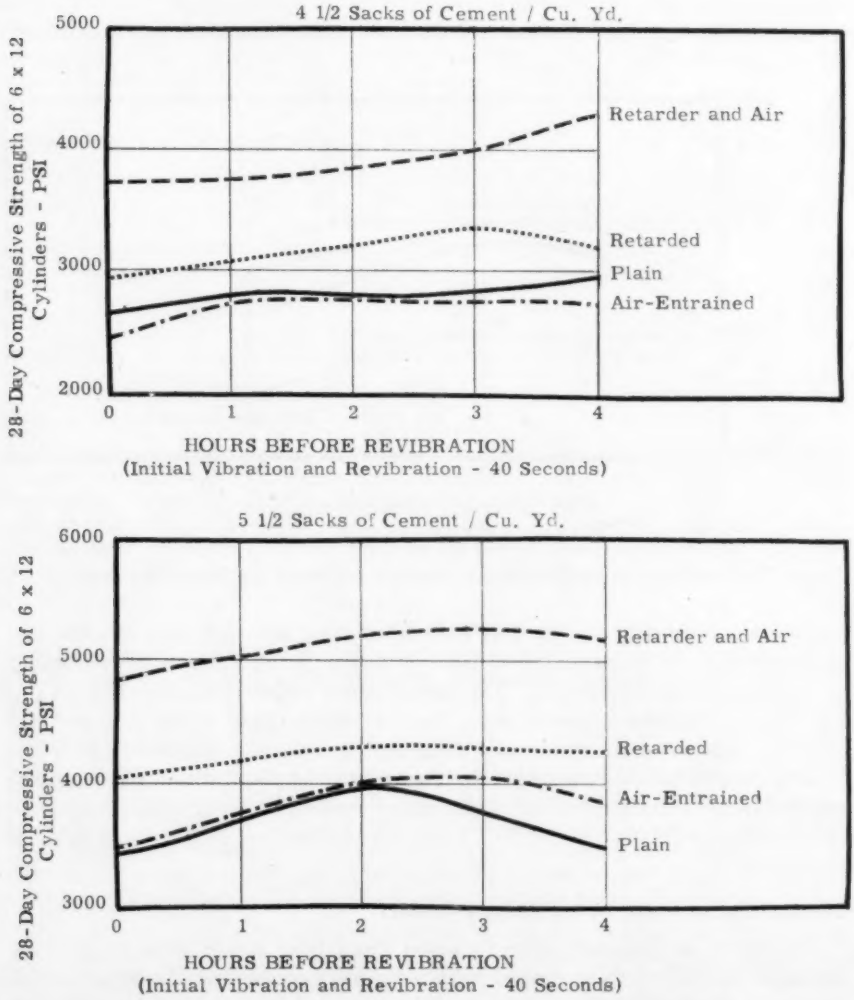

Fig. 2—Effect of revibration on compressive strength of laboratory-mixed concrete

Water requirement was reduced by the use of an air-entraining agent or a retarder. Air entrainment also improved the appearance of the somewhat harsh mixes used, and the combination of air entrainment and retarder resulted in much higher strengths than any of the other mixes.

Sawyer and Lee[3] reported maximum increase in 28-day compressive strength caused by revibration to be 24 percent for plain concrete and 13 percent for air-entrained concrete. They also noted an increase in flexural strength of revibrated concrete, but the increase was considerably less. In their procedure the vibration time was increased progressively after greater intervals of

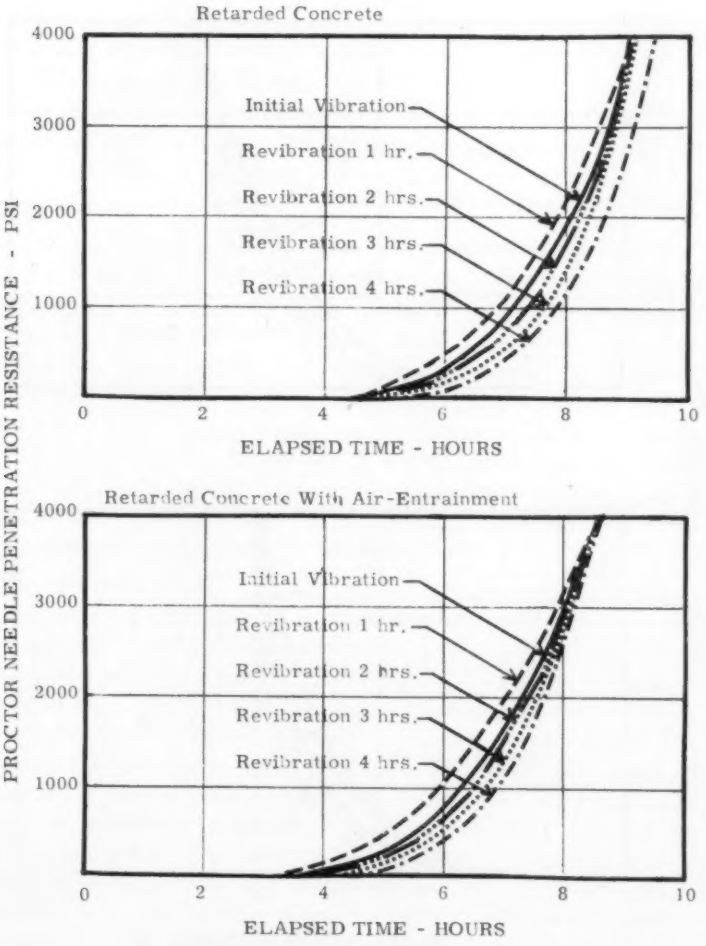

Fig. 3—Effect of revibration on final hardening time of retarded concrete

delay in order to obtain similar consistency of the concrete after revibration. Their specimens were revibrated for periods of 12 to 120 sec each.

M. F. Bastian[4] compared the strength of cubes which were vibrated once with the strength of cubes vibrated several times at later ages. A table vibrator was used in these experiments and the reported increase in compressive strength of specimens vibrated several times was 38 percent at 2 days, 27 percent at 3 days, 12 percent at 5 days, and 10 percent at 28 days. Flexural strength of prisms was also increased.

Tests reported in this paper show that concrete revibrated up to 4 hr after placing has higher compressive strength than concrete vibrated the same

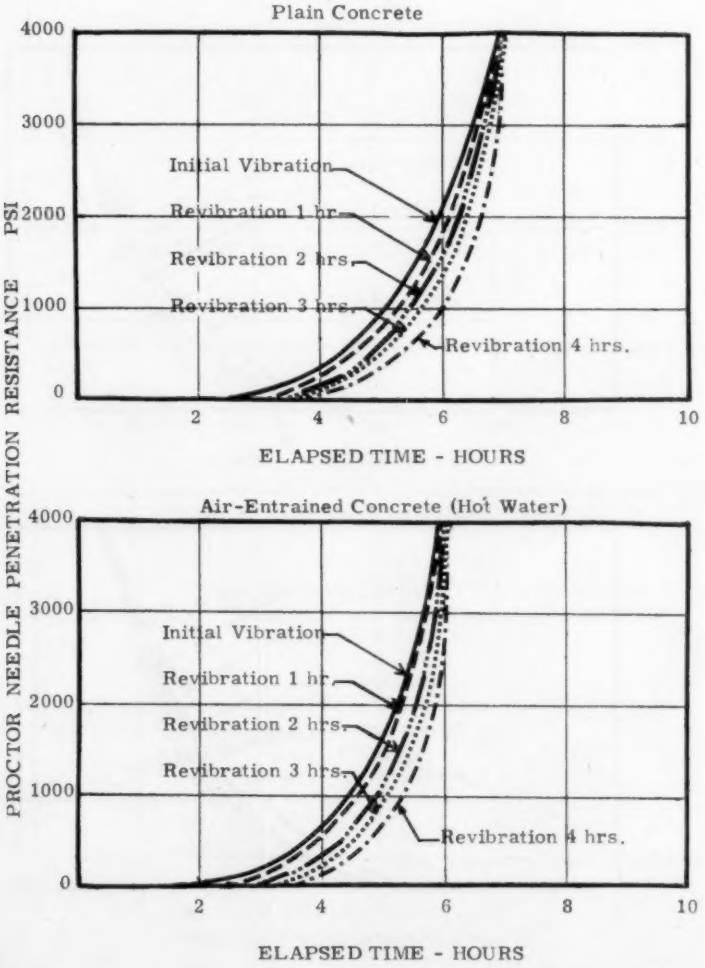

Fig. 4—Effect of revibration on final hardening time of plain or air-entrained concrete

length of time initially. If the period of revibration were increased to make the concrete plastic and similar in consistency to the initially vibrated concrete, the strength increase might be even greater.

Hardening time of mortar

Proctor needle penetration resistances of the mortars wet-screened from the various concretes are plotted in Fig. 3 and 4. Revibration made the mortars more plastic at delayed periods and penetration resistance was reduced immediately after vibration. The initial set of retarded concrete and concrete made with both retarder and air entrainment, was slightly accelerated by revibration at 1 hr, and the time required to reach 500 psi was reduced approximately 30 min. Revibration after 1 hr increased the time required to obtain a penetration resistance of 500 psi. Revibration of plain or air-entrained concrete at any period increased the time required to reach 500 psi. Maximum delay was approximately 1 hr.

Final hardening time is considered to be the time required to reach a penetration resistance of 4000 psi. Revibration did not have a significant effect on the final hardening time of any of the mixes.

Bleeding of concrete

Bleed water removed from the specimens was calculated as a percentage of original mixing water. Results of the bleeding tests are plotted in Fig. 5.

Specimens that were vibrated 40 sec initially had less bleeding than specimens vibrated only 20 sec initially and vibrated an additional 20 sec after 1, 2, 3, or 4 hr. These results are similar to results published by Higginson,[5] who found that increased vibration reduced bleeding.

Air-entrained concrete had a greater percentage increase in bleeding due to revibration than plain or retarded concrete. Increases in bleeding appeared to be accompanied by increases in compressive strength. In general, delays in revibration that resulted in maximum bleeding also resulted in maximum increases in compressive strength.

Uniformity of concrete

The influence of revibration on uniformity of concrete was considered. It could be reasoned that revibration could either cause heavy materials to settle to the bottom of a concrete lift and cause more paste to rise to the surface, or it could bring the cement and aggregate into more intimate contact and distribute the ingredients more uniformly throughout the concrete.

In order to determine the effect of revibration on uniformity as indicated by strength near the top and bottom of the blocks previously made and tested with the impact hammer, apparent average strength 2 in. from the top and 2 in. from the bottom of each block was calculated. The standard deviation and coefficient of variation was also calculated from all of the strength readings made at 2-in. intervals from top to bottom on the face of each block.

Test results are shown in Table 3. Revibration does not appear to influence the uniformity significantly. All blocks that were only vibrated initially

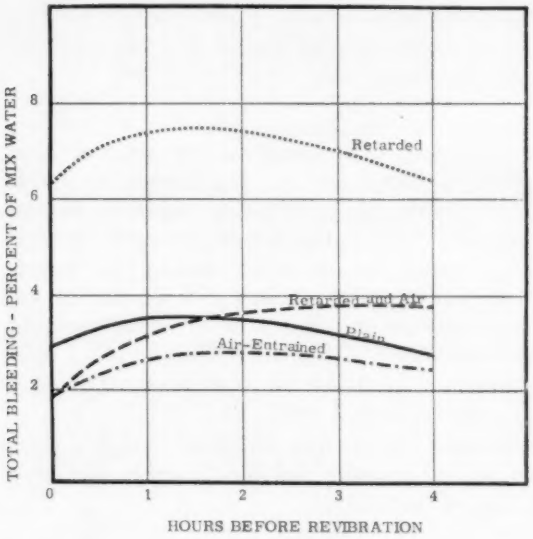

Fig. 5—Effect of revibration on bleeding of concrete

had higher strength at the bottom than at the top. Ten of the 16 blocks that were revibrated 1 to 4 hr after placing had higher strength at the top and one block had the same strength at top and bottom.

The coefficient of variation of revibrated plain concrete is less than the coefficient of variation of similar concrete vibrated only once initially, indicating the uniformity was improved by revibration. However, revibration increased the coefficient of variation of both air-entrained concrete and retarded concrete. The uniformity, as measured by the coefficient of variation of concrete that contained both the retarder and entrained air, was essentially unchanged.

After tests were completed with the impact hammer, 2 x 2 x 24-in. bars were cut from the face of each block with a concrete saw to observe the distribution of aggregate on the cut surfaces. Examination of all bars indicated revibration does not cause segregation or influence aggregate distribution.

Appearance of concrete surfaces

A box 24 in. high and 12 in. square was fitted with a glass plate on one side in order to observe behavior of concrete during vibration and revibration. The box was filled with concrete in one lift and vibrated by two 20-sec insertions of the laboratory vibrator, one insertion in the back corner, one in the front corner. This amount of vibration consolidated the concrete and removed most of the voids and pockets appearing against the surface of the glass. After the vibrator was removed, water began to collect in the voids remaining against the surface, and some pockets appeared to enlarge. Revibration 1 hr later in the same locations previously used removed some of the entrapped

TABLE 3—UNIFORMITY OF CONCRETE BLOCKS AS INDICATED BY IMPACT HAMMER

	Plain concrete					Air-entrained concrete				
Hr before revibration	0	1	2	3	4	0	1	2	3	4
Strength 2 in. from top, psi	4250	4300	4450	4800	3750	3650	4150	4300	3850	3300
Strength 2 in. from bottom, psi	4700	4800	5500	4700	4800	3700	4200	3850	3450	3550
Average strength of entire block, psi	4000	4650	5000	4500	4200	3710	3930	4070	3800	3470
Standard deviation of entire block, psi	207	142	176	162	154	53	124	176	140	161
Coefficient of variation, percent	5.2	3.1	3.5	3.6	3.7	1.4	3.1	4.3	3.7	4.6

	Retarded concrete					Retarded concrete with air entrainment				
Hr before revibration	0	1	2	3	4	0	1	2	3	4
Strength 2 in. from top, psi	4300	6100	4400	4800	3900	4650	4750	5000	4700	4600
Strength 2 in. from bottom, psi	4900	4950	4600	5700	5400	5200	4150	5200	4700	4700
Average strength of entire block, psi	4550	5200	4800	4880	4900	4500	4750	4900	4900	4600
Standard deviation of entire block, psi	163	228	207	223	331	147	127	155	207	128
Coefficient of variation, percent	3.6	4.4	4.3	4.6	6.8	3.3	2.7	3.2	4.2	2.8

water and improved the appearance of the concrete. Additional vibration after 2 and 3 hr improved the appearance of the concrete even more.

In another test, a harsh concrete mix having a 2-in. slump was placed in a box, vibrated initially, and revibrated after 1, 2, 3, and 4 hr. After the forms were removed, the concrete was honeycombed on the surface. Revibration may have been responsible for some improvement in the appearance of this concrete, but the appearance would have been considerably better with a more workable mix.

CONCLUSIONS

These conclusions are based on the results of tests performed on mixes discussed in the paper, in which revibrated concrete was vibrated the same total length of time as concrete vibrated only once. Different combinations of materials and increased vibration or revibration may produce different results.

1. Concrete that is to be revibrated must be properly proportioned with suitable materials. All other factors relating to good concrete practice must be followed.

2. Revibration increases the 28-day strength of concrete. Average maximum increase in strength is 13.8 percent.

3. Revibration increases the apparent strength indicated by the impact hammer at 28 days. Maximum strength gain is obtained when the concrete is revibrated 1 or 2 hr after placing.

4. Revibration up to 4 hr after placing has no effect on the final hardening time of concrete as measured by a reading of 4000 psi with the Proctor needle. Revibration may increase the time required to obtain a reading of 500 psi on the Proctor needle as much as 1 hr.

5. Revibrated concrete bleeds more than concrete given the same total amount of vibration initially.

6. Revibration does not cause segregation, but it has no significant effect on uniformity of compressive strength measured at 2-in. intervals from top to bottom of 24-in. block.

7. Appearance of concrete can be improved by revibration.

8. The delay between placement and revibration of concrete can be increased if a set retarder is added to the concrete.

9. The delay between placement and revibration that is necessary to increase compressive strength of concrete is less critical if a set retarder is added to the concrete.

ACKNOWLEDGMENT

The tests were made in the laboratory of Sika Chemical Corp., Passaic, N. J. The writer is indebted to Emil Schmid for valuable criticisms and to the laboratory personnel for valuable assistance in conducting the tests.

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Discussion of this report should reach ACI headquarters in triplicate by June 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

Concrete for Sewage Works*

By E. C. WENGER†

SYNOPSIS

Concrete for sewage works and the control of concrete corrosions therein is fully discussed. Characteristics and treatment of sewage are briefly reviewed, along with the conditions of exposure of concrete in such construction. Quality of concrete, construction practices, and other steps necessary to minimize concrete corrosion are presented.

Concrete is an essential material for the construction of sewers and sewage treatment plants. Before discussing concrete for sewage works, however, it might be well to briefly review the characteristics of sewage and other conditions of exposure to which concrete is subjected in such construction.

SEWAGE CHARACTERISTICS

Sewage contains dissolved and floating substances of animal, vegetable, and mineral origin. The animal and vegetable substances, collectively called organic matter, are in large part offensive in character.

Biologically, sewage contains a vast number of living organisms among which bacteria predominate. These bacteria do the greater share of the work in sewage treatment plants by reducing the complex organic material in the sewage into simpler compounds.

Oxygen requirement

Bacteria may be classified as aerobic, anaerobic, and facultative. The aerobic bacteria require free or dissolved oxygen; the anaerobic bacteria thrive on oxygen contained in organic matter; the facultative bacteria are capable of thriving in varying amounts of oxygen, and may be aerobic or anaerobic.

Fresh sewage, because it contains a considerable amount of dissolved oxygen, is acted upon largely by aerobic and facultative bacteria. But unless additional free oxygen is absorbed from the atmosphere or is injected, as compressed air, into the flowing sewage, the oxygen supply will be soon depleted. When the free oxygen is entirely exhausted, aerobic bacteria activity ceases and only the anaerobic and facultative bacteria act upon the sewage. These bacteria produce putrefactive changes as proteins, starches, sugars, and other organic compounds are broken up into simpler substances. At the same time, methane, carbon dioxide, hydrogen sulfide, nitrogen, and other gases are generated.

*Presented at the ACI 53rd annual convention, Dallas, Tex., Feb. 27, 1957. Title No. 54-40 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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As sewage flows through the sewer conduit, it absorbs oxygen from the air in the conduit above the flowing sewage. Since the rate of absorption of oxygen in flowing streams varies as the 1.75 power of the velocity,[8] adequate velocity of flow is essential as well as ventilation of the sewer conduit, if the sewage is to remain fresh during its flow to the sewage treatment plant.

Acid wastes

Municipal sewage is composed largely of domestic wastes combined with some industrial wastes. Domestic wastes are predominantly alkaline, as are industrial wastes, except in some few cases. Municipalities should, and many do, prohibit the discharge of acid wastes with a pH of less than 5.5 directly into any public sewer.[1] Generally, wastes with pH above 5.5 will be neutralized by the excess alkalinity of the domestic sewage. Strong acid wastes are objectional because they will corrode not only concrete, but also other materials—such as steel and iron—used in sewage works. Such acid wastes also interfere with the efficient operation of the sewage treatment plant, as excessive acidity is harmful to the biological sewage treatment processes. When this occurs, sewage is likely to become septic, resulting in noisome odors in the vicinity of the sewage treatment plant.

Where acid wastes of a pH of less than 5.5 are to be continuously discharged into a sewer, concrete should not be used without a protective lining or coating. However, an occasional discharge of acid of short duration will not appreciably affect the life of a quality concrete sewer structure.

HYDROGEN SULFIDE GENERATION AND CONTROL

Another possible source of corrosion in sewers is that occurring indirectly from hydrogen sulfide generation. Some engineers not familiar with the factors affecting hydrogen sulfide generation hesitate or refuse to use concrete in sanitary sewers, in many instances even where hydrogen sulfide is not a problem.

In most instances, hydrogen sulfide can be controlled in sewers by eliminating in design one or more of the several factors which must be present for its generation. Hydrogen sulfide is generated in the slimes and deposited sludge on submerged surfaces of the sewer when the dissolved oxygen is so depleted that biochemical activity begins to reduce sulfates to sulfides.[2]

Hydrogen sulfide hazards

Among the many reasons why hydrogen sulfide generation should not be tolerated in any sewer system are these:

1. Hydrogen sulfide has a nauseating odor which destroys property values along the line of the sewer.
2. It interferes with the efficient operation of a sewage treatment plant, also causing odors that have resulted in depreciation of property values in the vicinity of the plant.
3. It corrodes various materials and equipment used in sewage works.
4. It is a deadly gas. Sewer maintenance workmen and citizens have lost their lives from exposure to this gas.

Sewer design for hydrogen sulfide control

Hydrogen sulfide itself will not corrode concrete. However, after it is evolved as a gas into the sewer atmosphere, it may be oxidized on the walls of the sewer above the flowing sewage by aerobic bacteria to form droplets of sulfuric acid, which cause corrosion. In a gravity sewer, if sufficient oxygen is absorbed by the flowing sewage, there will be no sulfide buildup, no evolution of the gas, and consequently no corrosion.

In 1946, Pomeroy and Bowlus[4] stated a principal for free flowing sewers as follows: "For any particular temperature and sewage condition there is a limiting velocity of flow above which sulfide buildup will not occur." This velocity can be expressed by the formula:

$$V = \sqrt{\frac{(\text{B.O.D.}) \times (1.07)^{t-20}}{55}}$$

where B.O.D. is the strength of the sewage as determined by the biochemical oxygen test[3] and t is the temperature in degrees centigrade and V is the velocity of flow in ft per sec. For average conditions of strength and temperature of sewage, the velocity of flow required to prevent sulfide buildup in sewers is but slightly above the 2 ft per sec that is considered necessary for self-cleansing properties. In extreme conditions, velocities up to 4 ft per sec may be required.

Experience has shown that where velocities in gravity sewers are provided to satisfy the formula referred to above, no significant corrosion has resulted. Turbulence in sewage flow should be kept to a minimum, as it causes increasing evolution of the gas into the sewer atmosphere.

In a force main carrying sewage, no surface area is available for oxygen absorption. Consequently, there will be a buildup of dissolved sulfides, even if the sewage is weak. Since the pipe is flowing full, no evolution of the gas is possible, and consequently no corrosion will occur in the force main. However, at the end of the force main, if it discharges into a gravity sewer, hydrogen sulfide will be liberated and corrosion of the sewer structure at that point and for some distance downstream is likely. To prevent such corrosion several things may be done: ammonia may be injected into the sewer atmosphere to neutralize the sulfuric acid as it forms on the walls of the sewer; the generation of hydrogen sulfide can be inhibited by use of chemicals in the wet well at the pump station; or oxygen may be added by injecting compressed air at the pump discharge. The amount of air required will depend upon the amount of sulfide generated in the force main, which can be calculated knowing the strength and temperature of the sewage, and the inside surface area of the pipe.

Other control measures

Where conditions are such that hydrogen sulfide generation cannot be controlled in design, other measures can and should be taken to inhibit its generation. Among these are the use of chemicals, such as chlorine, sodium

nitrate, metallic salts, lime, and ammonia. Proper maintenance including periodic cleaning of sewers, to insure design velocities of flow, is also essential.

PLANT TREATMENT

Treatment of the sewage at the plant is arbitrarily divided in two parts, primary and secondary. These are matters of degree in which about one-third of the organic load is removed by the primary treatment, and an additional 50 to 60 percent by secondary treatment.

At the sewage treatment plant, both aerobic and anaerobic bacteria play an important part in the treatment process. Oxygen is introduced into the sewage to stimulate aerobic action in the aeration tanks and its presence is necessary in the trickling filters. In the sludge digestion tanks oxygen is kept out to induce septic action by anaerobic bacteria.

Normally, there are no reactions harmful to concrete as the sewage passes through the treatment plant, except in the initial operation of the sludge digesters where an acid fermentation may occur for a short period. Generally, alkaline fermentation rapidly establishes itself. Where acid fermentation persists, it may be necessary for a short time to inject into the digesters some well-digested sludge or to neutralize the acid by the use of lime until alkaline fermentation is established to prevent any corrosion of the concrete in contact with the sludge effluent.

In the operation of the digesters, many gases including hydrogen sulfide are generated. But, because of the absence of air in the digesters, oxidation of hydrogen sulfide to sulfuric acid cannot occur and no corrosion results.

CONCRETE FOR SEWERS AND TREATMENT WORKS

General requirements

The concrete in the sewage treatment plant structures is generally subjected to severe exposures of repeated cycles of freezing and thawing and wetting and drying. Therefore, it is essential that the concrete be of the highest quality. To attain this quality it is mandatory that these basic principles of concrete making be rigidly adhered to:

1. The use of sound, well-graded aggregates
2. Low water-cement ratio
3. Properly proportioned mix
4. Careful placement
5. Adequate curing

Air entrainment

Air entrainment should be added to these basic requirements. Air-entrained concrete was developed originally to improve the resistance of concrete to surface scaling resulting from salt application for ice removal. It has served this purpose well. In addition it has been found that air-entrained concrete has many other beneficial properties particularly desirable in concrete for sewers and sewage treatment works. These are:

1. Increased resistance to freezing and thawing, and wetting and drying
2. Increased workability and cohesiveness

3. Reduced segregation and bleeding
4. Reduced permeability and an increased resistance to sulfate waters

Because of the relatively thin walls in sewage treatment plants, it is sometimes difficult to place concrete around reinforcing steel and still obtain satisfactory consolidation. This difficulty can be overcome by the use of properly proportioned air-entrained concrete mixture of well-graded and suitably selected maximum sized aggregates, and the use of vibration in placing the concrete. Under some conditions admixtures have been used to increase the plasticity and placeability of the concrete; this should be acceptable where they have been shown to have no injurious effect on strength and durability.

Cement types used

Accelerated tests have indicated that in strong acid solutions (pH of 1 to 2) there is an apparent increase in resistance of air-entrained concrete made with Type II cement as compared with Type I. This has led many engineers to believe that Type II and all other cements low in tricalcium aluminate are acid resistant. Such is not the case. Concrete made with all types of portland cement, including Type II, react with acids, forming soluble calcium salts.

Corrosion of concrete by sulfuric acid can be considered in two stages:

1. Reaction of the acid with the lime in the cement forming calcium sulfate
2. Reaction of the calcium sulfate with the calcium aluminates in the cement, forming calcium sulfoaluminates

Type II cement, being a sulfate resisting cement, is not affected by the calcium sulfate and no further corrosion takes place until the calcium sulfate is removed, and a new surface of the concrete is exposed to the acid solution. However, concretes made of Type I or Type III cements, not being sulfate resistant, are attacked by the calcium sulfate, and corrosion continues uninterrupted during the second stage.

Tests[5,6] indicate that the use of certain pozzolanic admixtures with some Type I cements improves sulfate resistance compared with straight Type I cement, and provides sulfate resistance comparable to that attained by Type II cement. Similar tests indicate that use of pozzolanic materials with Type II cement does not improve its sulfate resistance.

Since sulfuric acid reacts with the lime in the cement, the rate of corrosion will vary with the amount of cement in the mix that is present to react with the acid. Therefore, the more cement, the slower the attack. Likewise, concrete made with dolomite or limestone aggregates should deteriorate more slowly than concrete made with granite or quartzite aggregates.[7] An extra inch or more of concrete added to sewer walls where sulfides cannot be controlled entirely will add appreciably to the ultimate life of the sewer.

CONCLUSION

The corrosion of concrete in sewage works, while admittedly a serious problem where it occurs, is actually a rare occurrence, as evidenced by the

many miles of concrete sewers and the large number of disposal plants that have been in satisfactory use for many years.

Corrosion of concrete in sewage works, where it occurs, depends on a combination of certain physical, chemical, and biological conditions which result in a complex chain reaction. If this chain is broken, no corrosion will occur. It is possible to break this chain by controlling certain design features such as:

1. Providing an adequate velocity of flow in the sewer for the strength and temperature of the sewage
2. Insuring proper ventilation of the sewer
3. Eliminating excessive turbulence in flow

Where conditions are such that these design requirements cannot be met, other economic means can be taken to break the chain such as:

1. The use of chemicals to inhibit certain reactions
2. The use of compressed air to keep sewage fresh
3. Periodic cleaning of sewers to remove slime and prevent obstructions from reducing velocity
4. The use of acid-resisting liners

Since the concrete in sewage works is subject to severe exposure, it is essential that it be of the highest quality. While no portland cement concrete is completely immune to strong acid attack, the higher the quality of the concrete, the greater its resistance to such attacks. The selection of suitable well-graded aggregates, a low water-cement ratio, air entrainment, careful placement, thorough consolidation, and adequate curing are necessary essentials for the quality of concrete required in sewage works.

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Discussion of this report should reach ACI headquarters in triplicate by June 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

Creep of Concrete under Variable Stress*

By A. D. ROSS†

SYNOPSIS

Creep has important effects on the stresses and deflections of reinforced and prestressed structures. Concrete structures in use are subject to variable stress whereas most of the data on creep have been obtained under constant stress. It is therefore desirable to be able to compute creep under variable stress from the results under constant stress.

Three methods for doing this are discussed and experiments designed to test these methods are described. Each method has a certain advantage in particular circumstances depending on the character of the stress variation, the extent of creep data available, and the accuracy required.

INTRODUCTION

Unstressed concrete shrinks in air and, under a sustained stress, an additional time-strain, creep, appears which may amount to several times the initial elastic strain. These time-dependent movements profoundly affect stresses and deflections of concrete structures.

Under no external load, shrinkage creates a system of stresses because the shrinkage tendency is resisted by reinforcing steel or by the interior parts of the concrete itself where contraction is less than at the surface. Creep greatly moderates the severity of these stresses. More usually, however, there is a permanently sustained load, and creep and shrinkage combine to affect the behavior of the structure. In particular, steel reinforcement located in compressed areas of beams and columns is subjected to severe stress increases which may reach the yield point of a mild steel. In pretensioned and post-tensioned concrete structures there is a gradual decline in prestress due to creep and shrinkage. Although the importance of this loss can be reduced by employing a high initial stress in the steel, it is still not negligible, and it places certain restrictions on the designer. Creep and shrinkage have important effects in statically indeterminate structures; if, for example, there is a movement of the abutments of a two-pinned arch when loaded, creep of the concrete will moderate stresses caused thereby and will tend to restore[1] conditions to those envisaged by the designer—an elastic structure with immovable abutments. On the other hand, shortening of the beam of a prestressed portal frame due to creep and shrinkage alters the horizontal reaction at the feet and thus creates secondary moments.

*Received by the Institute Feb. 11, 1957. Title No. 54-41 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1958. Address P.O. Box 4754, Redford Station, Detroit 19, Mich.

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Effects of creep and shrinkage are most influential in the "continuous" prestressed pavement. In this system, long concrete slabs are "gap-jacked" against each other and, at the terminal points, against fixed abutments. The strain thus created in the concrete is held with the object of securing a permanent compressive stress but, in practice, creep and shrinkage cause severe relaxation of the initial stress. It is therefore necessary to restore the losses of stress by jacking several times over a fairly long period; without this action the slab, despite its initial compression, would soon pass into tension, the condition which the system is designed to avoid.

Many other cases of the influence of creep and shrinkage on stresses and deflections of structures can be cited. However, time-dependent strains do not generally affect the ultimate load capacity, because the mechanism of failure is controlled by the relatively enormous strains which develop as collapse is approached, regardless of the previous strain history. Only in cases in which instability is a factor can creep materially affect the failing load. The buckling load of, say, a slender column is a function of the lateral displacement, and since this is altered by creep it follows that the available load factor of a permanently loaded column declines[1] with lapse of time.

CREEP UNDER VARYING STRESS

The effects of creep and shrinkage on structures in normal service are so important that it is obviously desirable to be able to compute these effects. In structural mechanics, shrinkage can generally be included in the calculations as a strain, independent of stress, similar to the effect of a permanent reduction in temperature. If the stress were constant, estimation of the effects of creep would present no problem, because this is precisely the condition under which most creep data have been obtained experimentally. In all structures in use, stress in the concrete varies with time, and the problem becomes that of predicting the creep under varying stress from the results of tests under constant stress.

The deformations of the concrete at any instant are defined as follows:

$$\text{total strain} = \text{elastic strain} + \text{creep} + \text{shrinkage} \quad \ldots\ldots\ldots\ldots (1)$$

There is always the question whether the elastic strain after a period of sustained load should be calculated on the basis of an increasing elastic modulus or on the original value at the time of first loading. The point is somewhat academic and, for constant stress, only a matter of definition. If the elastic strain under a constant sustained stress is assumed to diminish with time, we then merely assume that creep is increased by a corresponding amount to insure that the total strain is correct. Nor, under varying stress, is the precise interpretation of elastic strain very important, because changes in elastic strains due to increase in elastic modulus are generally small compared with the sum of the other quantities in Eq. (1). Similar discussions on whether creep is an accelerated shrinkage and whether it is theoretically correct to

deduce creep of a loaded specimen by subtracting shrinkage of an unloaded specimen will not be entered into here.

Of numerous factors affecting the magnitude of creep, only the effect of stress will be considered here; a general review of creep can be found in a valuable paper[3] published in 1947. Provided the stress does not exceed 0.4 to 0.5 of the ultimate strength, creep is approximately proportional to stress. This proportionality greatly simplifies all analyses and enables the creep per unit stress, the "specific creep," to be quoted.

One method for predicting the creep under a variable stress is by means of a rheological model composed of elastic springs and viscous dashpots. By postulating a viscoelastic model which simulates the behaviour of concrete, we can obtain the differential equation

$$f - \frac{c}{\alpha} - \frac{1}{\phi}\frac{dc}{dt} = 0 \qquad (2)$$

in which f is the stress, c the creep, t the time, and α and ϕ are constants determined by tests. This equation and several modified forms of it appropriate to a number of structural problems can be integrated to give solutions[2] to a wide variety of problems in reinforced and prestressed concrete in which the stress varies. The method takes account of a variable stress history and it can provide formal mathematical solutions even for abrupt changes of stress. In its simplest form, it usually gives answers which differ little from those calculated by an effective modulus, to be described later. After an infinite time, assuming that stress has reached a constant value, both methods give the identical result. In view of this and because the model method leads to rather complicated mathematical expressions, it is not likely to be much used in practice and will not be considered further here.

There remain three important methods for computing creep under varying stress, and these are briefly described.

Effective modulus method

In this method, normal procedures in structural mechanics are used but the elastic modulus is replaced by a reduced or *effective* modulus which allows for both elastic and creep strains. If E_c is the elastic modulus, then the effective modulus is defined as follows:

$$E_c' = \frac{E_c}{1 + c_1 E_c} \qquad (3)$$

or

$$E_c' = \frac{1}{\epsilon + c_1} \qquad (3a)$$

in which E_c' is the effective modulus, c_1 is the specific creep under 1 psi at the appropriate time, and ϵ is the elastic strain due to stresses of 1 psi.

As a simple example of the application of this method consider the case of an axially pretensioned concrete member such as a pole or lamp standard which is fabricated in a long-line stressing bed. The wires are first tensioned to an initial stress σ_i and the concrete units are then cast around them. When the concrete is strong enough, the wires are released from the anchorages and, through bond stresses at the ends, they act as springs compressing the concrete. In the central length of the unit where the conditions are uniform, the stresses are simply determined by considering forces and strains. First, by equating forces:

$$\text{tensile force in the steel} = \text{compressive force in the concrete} \quad (4)$$

$$\sigma_t A_s = f_t A_c \quad (4a)$$

$$\sigma_t = f_t / p \quad (4b)$$

where σ_t = stress in the steel after transfer of load
A_s = area of steel
f_t = stress in the concrete after transfer of load
A_c = area of concrete
p = steel ratio = A_s/A_c

Second, by equating strains:

$$\text{compressive strain in the concrete} = \textit{change} \text{ of strain in the steel} \quad (5)$$

$$\frac{f_t}{E_c} = \frac{\sigma_i - \sigma_t}{E_s} \quad (5a)$$

where E_c = elastic modulus of concrete
E_s = elastic modulus of steel

By substituting (4b) in (5a) the stress in the concrete after transfer can be written as

$$f_t = \frac{p\,\sigma_i}{1 + n\,p} \quad (6)$$

where the modular ratio $n = E_s/E_c$.

Now consider the conditions much later when shrinkage has reached a value s and creep has reduced the elastic modulus E_c to the effective modulus E_c'. The modular ratio n is now increased to an effective modular ratio $n' = E_s/E_c'$. Because of creep and shrinkage, stress in the steel declines from its value σ_t at transfer to a lower value σ and concrete stress correspondingly declines from f_t to f. These stresses are computed, as before, by equating forces and strains.

$$\text{tensile force in the steel} = \text{compressive force in the concrete} \quad \ldots\ldots (7)$$

$$\sigma A_s = f A_c \quad \ldots\ldots (7a)$$

$$\sigma = f/p \quad \ldots\ldots (7b)$$

$$\text{total compressive strain in the concrete, elastic plus creep} + \text{shrinkage} = \text{total change of strain in the steel} \quad \ldots\ldots (8)$$

$$\frac{f}{E_c'} + s = \frac{\sigma_i - \sigma}{E_s} \quad \ldots\ldots (8a)$$

Combining (7b) and (8a) leads to

$$f = \frac{p\,(\sigma_i - s\,E_s)}{1 + n'p} \quad \ldots\ldots (9)$$

Thus, provided creep and shrinkage are known along with the other relevant information, the residual stress f remaining in the concrete and, of course, the stress σ in the steel can be readily computed. Many problems in structural mechanics can be solved by methods similar to those illustrated in this example.

The effective modulus is simple and convenient but, for varying stress, is theoretically unsound. The ultimate strain predicted depends only on ultimate stress and the method disregards stress history. It predicts complete recovery of strain if the stress is reduced to zero and this is not true of concrete. In many practical cases of sustained loading in structures, stress in concrete declines in course of time due to transfer to the steel, and under these conditions the effective modulus obviously will underestimate the strain after long periods. Limitations of the method will be discussed later.

Rate of creep method

From a specific creep curve the rate of creep dc_1/dt at any time t is known and if, at this time, the stress in a structural member is f, the increment of creep in the element of time dt is assumed to be $f dc_1$. Creep under a variable stress after a time t is therefore:

$$c = \int_0^t f \frac{dc_1}{dt}\, dt \quad \ldots\ldots (10)$$

This method attempts to avoid some weaknesses of the effective modulus method by integrating all elementary increments of creep, each of which is estimated from the unit-stress creep curve and the particular stress acting during the relevant element of time. In this sense it takes account of stress history, but it cannot be considered theoretically sound because of this. If,

after a loaded period, the stress is removed, then $f\,dc_1/dt$ is zero which means that the method predicts no time-dependent recovery. Concrete, however, definitely exhibits some recovery of creep. Again, in another sense, the method disregards stress history because it assumes that concrete will creep at the rate of $f\,dc_1/dt$ regardless of whether it was stressed severely or negligibly in its earlier history. For this reason the method may be expected to overestimate creep under a declining stress and vice versa.

To illustrate the application of this method, consider the foregoing example treated by the effective modulus method. After release of the wires, creep of concrete will alter the strains and reduce stresses in an element of time dt as follows:

elastic recovery of steel + elastic recovery of concrete = creep of concrete..(11)

$$-\frac{d\sigma}{E_s}-\frac{df}{E_c}=f\,dc_1 \qquad (11a)$$

in which σ is the stress in the steel.

Noting that $d\sigma = (1/p)\,df$, Eq. (11a) can be integrated to give:

$$f=f_t\,e^{-\theta c_1} \qquad (12)$$

in which f_t is the stress in the concrete immediately after transfer (see Eq. 6), e is the Naperian base and

$$\theta=\frac{p\,E_s}{1+np}$$

The solution can be modified to allow also for shrinkage which changes Eq. (11a) to:

$$-\frac{d\sigma}{E_s}-\frac{df}{E_c}=f\,dc_1+ds \qquad (13)$$

in which ds is the element of shrinkage in time dt. If we assume that shrinkage-time and creep-time curves have a similar form, we can write $ds = k\,dc_1$ in which k is determined from tests. Eq. (12) then becomes:

$$f=(f_t+k)\,e^{-\theta c_1}-k \qquad (14)$$

Thus the residual stress f in the concrete after any time can be calculated, and this solution is typical of many which can be written for a variety of structural problems using the rate of creep method. The deficiencies of this method have already been mentioned and its scientific appearance should

not encourage a belief that it is necessarily superior to the effective modulus method. The two methods give almost the same answer for the pretensioned concrete problem. For example, consider a numerical application of the previous equations with the following typical data:

$$\sigma_i = 150{,}000 \text{ psi} \qquad E_s = 28 \times 10^6 \text{ psi}$$

$$p = \frac{A_s}{A_c} = 0.007 \qquad E_c = 4 \times 10^6 \text{ psi}$$

$$c_1 = 0.3 \times 10^{-6} \qquad \varepsilon = 400 \times 10^{-6}$$

The stress in the concrete on release of the wires, by Eq. (6), is $f_t = 1001$ psi, and the residual stress f after creep and shrinkage have occurred is: (a) by Eq. (9), effective modulus method, 877 psi and (b) by Eq. (14), rate of creep method, 874 psi.

This difference in the calculated stress is, of course, quite unimportant; it represents the difference between the sum of the first two terms of the expansion of $e^{-\theta c_1}$ (which give the effective modulus solution) and the sum of all the terms. The third and subsequent terms are negligible for practical purposes. Despite publication of calculations giving the loss of stress by the rate of creep method, the effective modulus method is preferable because it is simpler and more convenient and because it gives practically the same answer. There is no justification for assuming that the rate of creep method is more accurate; both methods are based on approximations and, under a gradually declining stress, the truth is likely to lie between the two. The relative merits for other stress conditions are considered later.

Method of superposition

If one of three identical specimens is subjected to a sustained positive or compressive stress $+f$ applied at time t_0, its strain history would be as shown in Fig. 1(a). A negative or tensile stress of $-f$ is applied to the second specimen at time t_1 and its strain history is shown in Fig. 1(b). The third specimen is stressed to $+f$ at t_0 and $-f$ at t_1 (which is equivalent to unloading) and its strain history is shown in Fig. 1(c). Now if, on adding the experimentally determined diagrams (a) and (b), a diagram (c) is obtained which matches the experimental result, the material is said to conform to the principle of superposition.

This example is a special case since it assumes the application of the full reversed stress at t_1. However, since creep is proportional to stress, it will be evident that the effects of a variation of stress of any magnitude (within the proportional range) and in either direction can be estimated by superposing the appropriate creep curves, provided the principle is applicable. Note that the creeps in tension and compression are assumed equal for equal stresses, which is approximately true of concrete in average humidities. Some plastics conform fairly closely to the principle as illustrated in Fig. 1 but concrete certainly does not, because it never shows a complete recovery

and the ultimate permanent set may be a high proportion of the creep. McHenry[4] considered this due to the changing properties of concrete with increasing age, and he restated the principle of superposition, making suitable allowance for the reduced creep under later increments of stress. This has the effect shown dotted in Fig. 1, and McHenry reported that concrete obeyed the principle of superposition in this modified form.

Not all stress variations are step functions of time as shown in Fig. 1, but obviously it is always possible to handle a continuous variation as a series of small but finite increments. Using the principle of superposition it is not easy to write a general algebraic solution to a problem in the manner of Eq. (9) and (14) for the pretensioned member, but a numerical solution to a particular case can always be obtained.

TESTS OF CONCRETE

The three methods discussed above are so fundamentally different that interest is aroused in their relative accuracies; only by tests can they be reliably compared. In such tests, actual structural members such as reinforced columns, or beams, or pretensioned elements could be used; but it was considered preferable to submit plain concrete specimens to controlled and precisely known stress variations. Thus actual strains and those predicted by the various methods can be compared without making any assumptions about composite action with the steel in a practical member.

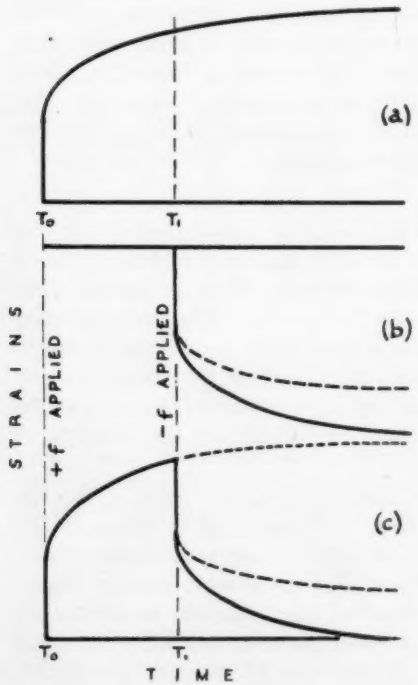

Fig. 1—Application of the principle of superposition to time-dependent strains

General conditions

The early researches on creep provided much data for low and medium strength concretes, but information on high strength concretes has been scanty. Although the present tests were planned primarily as a study of creep under varying stress, the opportunity was taken to obtain creep data for a concrete having a strength suitable for prestressed construction.

A 1:1.6:2.8 mix with rapid hardening portland cement and a water-cement ratio of 0.375 by weight was compacted by vibration; this concrete had a 4 in. cube strength of 9600 psi after 28 days at 17 C. The

creep specimens were 4⅝ in. diameter cylinders 12 in. long; the strength of the concrete in this form was 70 percent of the cube strength. A simple stressing device was developed to sustain loads up to 17 tons (2265 psi on the concrete) and each test rig incorporated a load dynamometer in the form of a length of mild steel tubing with provision for strain measurements along three gage lengths disposed at 120-deg intervals around the perimeter. Every loaded specimen was accompanied by an unstressed control to establish the shrinkage under identical atmospheric conditions. All specimens were stored in a chamber maintained at 17 C and 93 percent relative humidity.

Tests to establish creep under constant stress

Specimens were subjected to constant sustained stresses applied at various ages. Tests at various stresses up to 2000 psi once again confirmed that creep is approximately proportional to stress and therefore the results, after deduction of shrinkage, are presented in Fig. 2 as total strains per unit stress. Each curve represents the mean of two, three, or more tests. The curve for loading at 120 days was extrapolated; other creep curves for loading at intermediate ages were interpolated as required by the later calculations.

Compared with other published data the creep values shown in Fig. 2 are low because of the high strength of the concrete, and particularly because of the high humidity. For example, the specific creep after 170 days when loaded at 28 days is only about 0.12×10^{-6}. Specimens of the same concrete

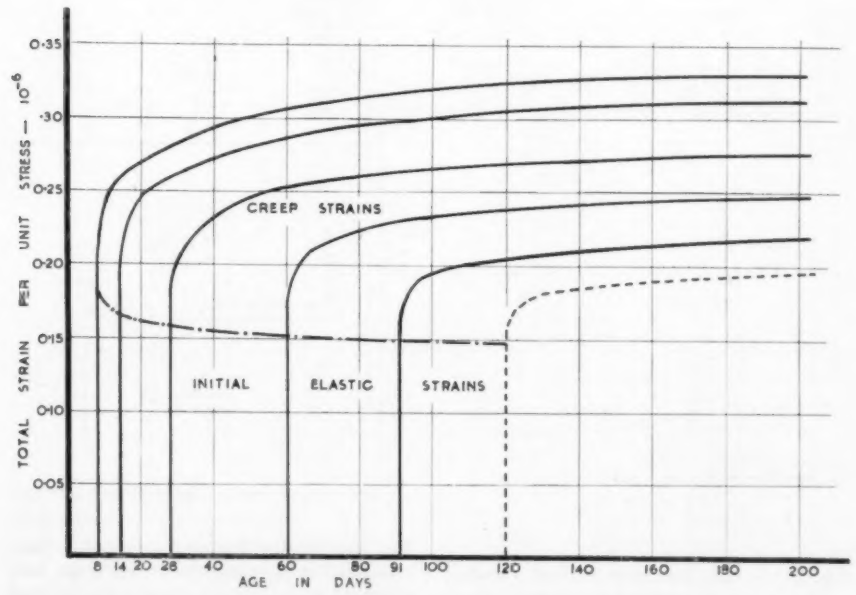

Fig. 2—Total strains due to constant sustained stresses applied at various ages

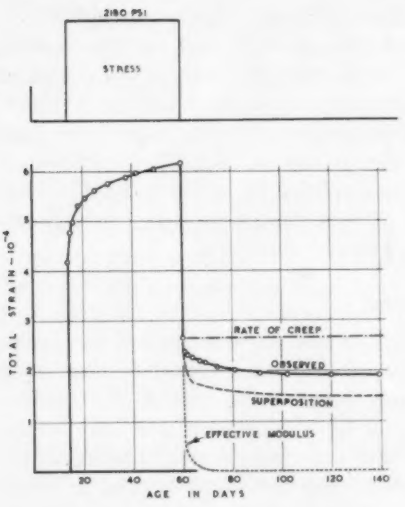

Fig. 3—Strains during and after limited period of constant sustained stress. Loading period 14 to 60 days

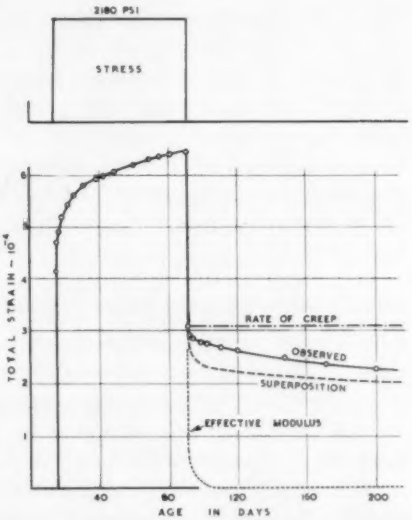

Fig. 4—Strains during and after limited period of constant sustained stress. Loading period 14 to 91 days

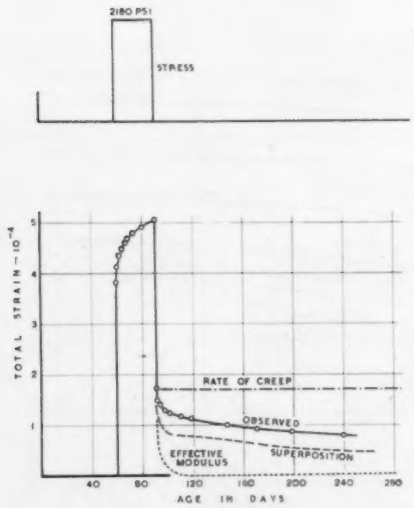

Fig. 5—Strains during and after a limited period of constant sustained stress. Loading period 60 to 91 days

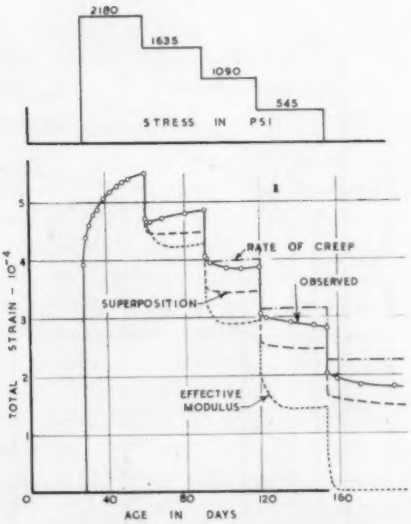

Fig. 6—Strains during a variable (decreasing) stress program. The stress was reduced from a high initial value in four decrements during a period of 126 days

maintained under stress in uncontrolled winter conditions out-of-doors in London have shown about 50 percent more creep. In summer creep would be still greater.

Tests under severe variations of stress

The first three tests (Fig. 3, 4, and 5) were designed to examine the simplest case of stress variation, i.e. a period of constant stress followed by complete removal of load so that recovery of creep under zero stress could be observed. In the next six tests (Fig. 6-11), stress was altered in various steps, sometimes increasing, sometimes decreasing, and with both increases and decreases. It was expected that these variations would provide a critical test of the methods of calculating creep strains. In all these diagrams the stress variations are shown diagrammatically above the strains and zero time represents the time of casting. Shrinkage, as determined by the controls, although very small because of the high humidity, has been deducted from all observed movements so that the plotted strains are those due to stress alone. Four strain-time records are shown in each case as follows:

1. The observed experimental record.
2. A record computed from values of the effective modulus which were obtained from the appropriate test under constant stress, commenced at the corresponding initial age.

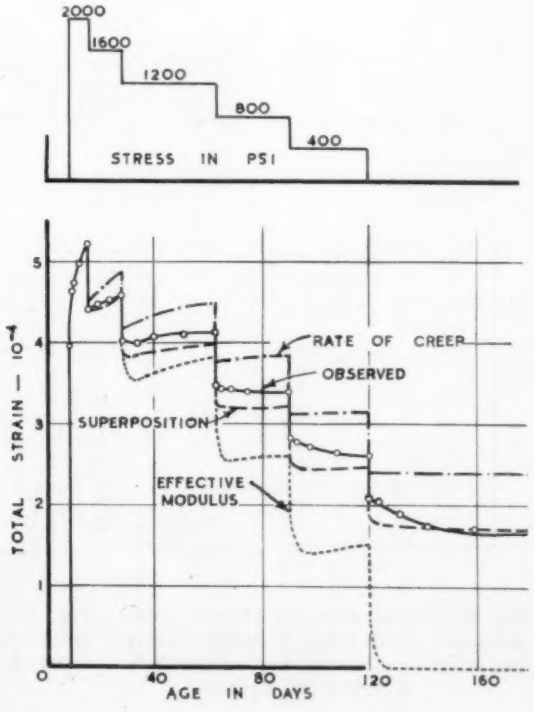

Fig. 7—Strains during a variable (decreasing) stress program. The stress was reduced from a high initial value in five decrements during period of 113 days

3. A record computed by the rate of creep method using values from the test under constant stress commenced at the corresponding initial age. All changes in elastic strain, at whatever age, have been calculated according to the elastic modulus at first loading. This might be criticized, but although the data were sufficient to enable suitable adjustments to be made for an increasing modulus, it was felt desirable to adhere to a single value of the elastic modulus when applying the method to these experiments because a structural problem, such as that on p. 745 would be handled thus.

4. A record computed by the theory of superposition using the various creep curves and elastic strains appropriate to each increment of stress at various ages. Interpolated values were obtained, where necessary, from Fig. 2.

Under the discontinuous and arbitrary variations of stress in these tests it is obviously not possible to write general mathematical expressions for the strains as in the practical example discussed [Eq. (5a) and (11a)]. The reliability of the three theories can, however, be checked against the experimental results since the basic principles can be applied in numerical calculations which are analogous to the formal mathematical treatments. The calculations are conveniently set out in tabular form, and, to illustrate the procedure, Tables 1, 2, and 3 show the figures from which the theoretical curves in Fig. 6 are plotted and compared with the experimental record.

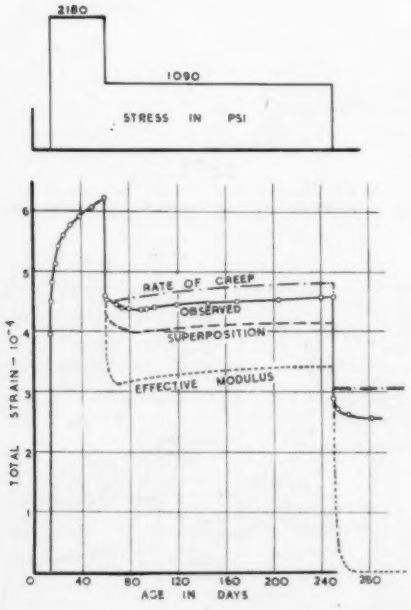

Fig. 8—Strains during a variable stress program. After a high stress for 46 days the stress was halved and sustained for a further 191 days

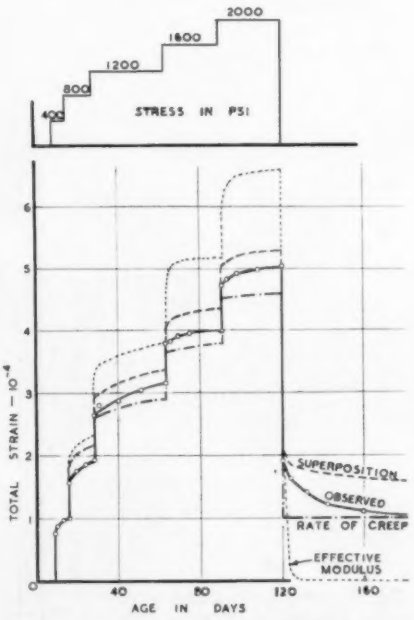

Fig. 9—Strains during variable stress program. The stress was increased to a maximum in five increments during a period of 112 days

TABLE 1—CALCULATION OF STRAINS UNDER VARIABLE STRESS BY EFFECTIVE MODULUS METHOD

Applied stress f, psi	2180					1635						1090				545				Nil	
Age, days	28	31	38	49	60	60	63	71	83	89	91	91	102	110	120	120	130	147	154	154	170
*Total strain $(\epsilon + c_1)$ under unit stress, $\div 10^{-7}$	1.61	2.10	2.34	2.44	2.51	2.51	2.53	2.57	2.61	2.63	2.635	2.635	2.65	2.67	2.68	2.68	2.70	2.72	2.73	2.73	2.74
Effective modulus $E_c' = \frac{1}{\epsilon + c_1}$, psi $\div 10^6$	6.21	4.76	4.28	4.10	3.99	3.99	3.95	3.89	3.84	3.80	3.79	3.79	3.78	3.75	3.73	3.73	3.71	3.68	3.66	3.66	3.65
Calculated strain $\frac{f}{E_c'} \div 10^{-6}$	3.51	4.58	5.10	5.32	5.47	?	4.14	4.20	4.26	4.30	4.31	?	2.88	2.91	2.92	?	1.47	1.48	1.49	?	0

*Values taken from curve in Fig. 2 for 28-day loading.

TABLE 2—CALCULATION OF STRAINS UNDER VARIABLE STRESS BY RATE OF CREEP METHOD

Applied stress f, psi	2180					1635						1090			545			Nil	
*Elastic strain $f \times \epsilon \div 10^{-7}$	3510					2632						1750			875			0	
Age, days	28	31	38	49	60	60	63	71	83	89	91	91	97	120	120	143	154	154	170
†Specific creep $c_1 \div 10^{-8}$	0	4.9	7.3	8.3	9.0	9.0	9.2	9.6	10.0	10.2	10.25	10.25	10.4	10.7	10.7	10.9	11.2	11.2	11.3
Increment of specific creep $\delta c_1 \div 10^{-8}$	0	4.9	2.4	1.0	0.7	—	0.2	0.4	0.4	0.2	0.05	—	0.15	0.3	—	0.2	0.3	—	0.1
Increment of creep $f \times \delta c_1 \div 10^{-7}$	0	1068	523	218	153	—	33	65	65	33	8	—	16	33	—	11	16	—	0
Total cumulative creep $\div 10^{-7}$	0	1068	1591	1809	1962	1962	1995	2060	2125	2158	2166	2166	2182	2215	2215	2226	2242	2242	2242
Total calculated strain $\div 10^{-6}$	3.51	4.58	5.10	5.32	5.47	4.59	4.63	4.69	4.76	4.79	4.80	3.92	3.93	3.97	3.09	3.10	3.12	2.24	2.24

*The initial value of $E_c = 1/\epsilon$ is assumed to remain unchanged throughout the period.
†Values taken from Fig. 2 for 28-day loading and excluding initial elastic strain $\epsilon = 0.161 \times 10^{-6}$ per 1 psi.

In Table 1 the total strains are simply calculated by dividing the stress at any instant by the appropriate value of the effective modulus as determined from a test at constant stress. There is an inconsistency, however, after an abrupt change of stress since we know that the immediate *change* of strain corresponds not to the effective modulus but to the elastic modulus. Thus, the rigid application of the effective modulus would predict an obviously erroneous value of strain immediately after a sudden change of stress as indicated in Table 1 by a question mark. This is one of the weaknesses of the effective modulus method. In the diagrams, the immediate effect of an abrupt change of stress is plotted as an elastic strain and the curve for a few days following is sketched until it joins with the later values calculated by effective modulus.

In Table 2 time is divided into elementary periods and creep during a period is calculated by multiplying the increment of specific creep by the stress prevailing during that period. Total strain at any instant is obtained by adding the elastic strain to the cumulative creep up to that instant. Table 3 requires little explanation since it is the numerical counterpart of Fig. 1. A specimen loaded at 28 days would, if subjected to a constant stress of 2180 psi, creep as shown in Column 3. At 60 days, however, a negative (tensile) stress of

TABLE 3—CALCULATION OF STRAINS UNDER VARIABLE STRESS BY METHOD OF SUPERPOSITION

Applied stress f, psi	Age, days	*Total strains $f(\epsilon + c_1)$ due to $f = 2180$ psi applied at 28 days $\div 10^{-6}$	†Deduct strains due to $f = 545$ psi applied at 60 days $\div 10^{-6}$	‡Deduct strains due due to $f = 545$ psi applied at 91 days $\div 10^{-6}$	§Deduct strains due to $f = 545$ psi applied at 120 days $\div 10^{-6}$	§Deduct strains due to $f = 545$ psi applied at 154 days $\div 10^{-6}$	Total calculated strain $\div 10^{-6}$
2180	28	3.51	—	—	—	—	3.51
	31	4.58	—	—	—	—	4.58
	38	5.10	—	—	—	—	5.10
	49	5.32	—	—	—	—	5.32
	60	5.47	—	—	—	—	5.47
1635	60	5.47	0.95	—	—	—	4.52
	62	5.52	1.08	—	—	—	4.44
	69	5.57	1.15	—	—	—	4.42
	81	5.65	1.22	—	—	—	4.43
	91	5.71	1.24	—	—	—	4.47
1090	91	5.71	1.24	0.89	—	—	3.58
	94	5.73	1.26	1.01	—	—	3.46
	102	5.77	1.27	1.07	—	—	3.43
	110	5.81	1.28	1.09	—	—	3.44
	120	5.83	1.29	1.10	—	—	3.44
545	120	5.83	1.29	1.10	0.86	—	2.58
	122	5.86	1.30	1.11	0.94	—	2.51
	147	5.93	1.32	1.14	1.02	—	2.45
	154	5.95	1.32	1.15	1.03	—	2.45
Nil	154	5.95	1.32	1.15	1.03	0.79	1.66
	159	5.95	1.33	1.16	1.04	0.85	1.51
	189	5.99	1.34	1.18	1.06	0.94	1.47

*$(\epsilon + c_1)$ values obtained from 28-day loading curve in Fig. 2.
†$(\epsilon + c_1)$ values obtained from 60-day loading curve in Fig. 2.
‡$(\epsilon + c_1)$ values obtained from 91-day loading curve in Fig. 2.
§$(\epsilon + c_1)$ values extrapolated from Fig. 2.

545 psi is applied and strains due to this stress—with proper allowance for aging—are deducted from the first column of strains. This procedure is repeated for the other decrements and final strain is obtained by superposing all the effects.

The three tables show the same total calculated strains as, of course they should, up to 60 days when the first variation of stress took place. It was noted earlier that when the stress remained constant there was no problem in calculating strains since our creep data were generally obtained from constant stress tests and that was the case initially in the present tests.

These numerical procedures can be applied whatever the nature of the variation of stress. If the stress varies continuously but not in accordance with any convenient mathematical law, the strains can be computed by the rate of creep method by considering short finite elements of time during which stress may be assumed to be constant and then applying the method indicated in Table 2. By the method of superposition, numerous small increments or decrements of stress must be considered and although the arithmetic is lengthened, no additional difficulty is created by a continuous stress variation. Using the effective modulus no difficulty arises since the strain at any time is estimated simply by dividing the stress by the appropriate value of the modulus.

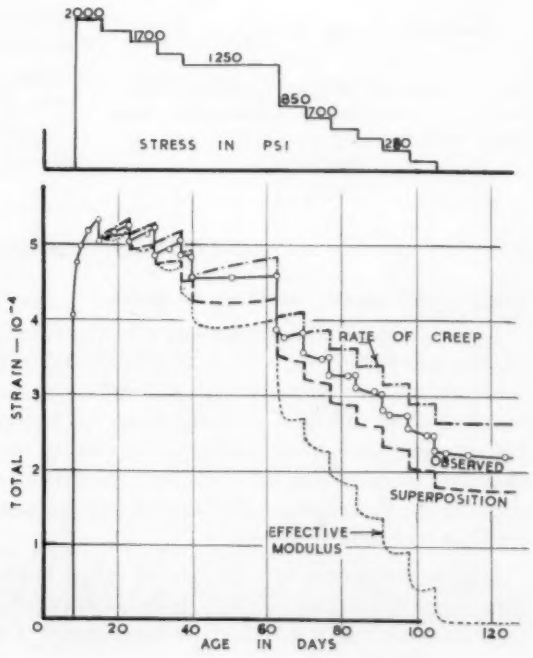

Fig. 10—Strains during program of decreasing stress. The stress was reduced in numerous decrements during a period of 112 days

Tests of relaxation of stress under constant strain

As an example of a continuous stress variation, specimens were stressed to 2000 and 1500 psi and thereafter the initial elastic strain was maintained constant by periodically adjusting the load. Thus, in these tests, the concrete was creeping under a variable declining stress. In order to isolate the effects of creep, the strain in the loaded specimen was not held quite constant but was allowed to vary by the small amount of the shrinkage measured on a companion control specimen. The recorded relaxation of stress is, therefore, due to creep alone.

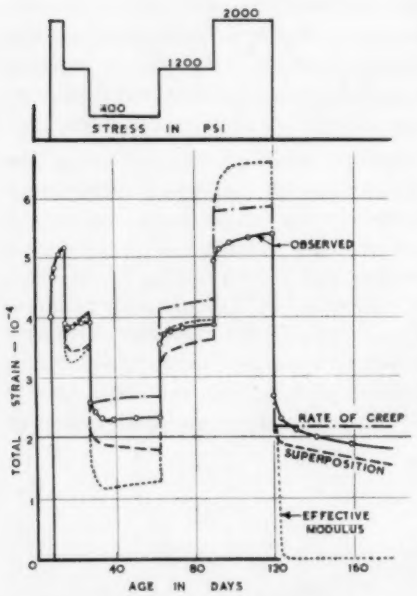

Fig. 11—Strains during variable stress program. From a high initial value, the stress was first decreased and then increased

The observed and computed results are shown in Fig. 12 and 13. Here, the effective modulus result was calculated as $\Delta E_c'$ in which Δ is the sustained strain and E_c' the declining effective modulus. The rate of creep result was calculated[5] from $f_i\, e^{-c_1 E_c}$ in which f_i is the initial stress and e the Naperian base. The superposition result was obtained by considering short but finite intervals of time and so adjusting the increments of stress that the summation of all elastic and creep strain increments remained constant.

DISCUSSION OF RESULTS

Creep under severe variations of stress

Fig. 3-11 show, as anticipated, that the effective modulus gives a poor prediction for severe variations, underestimating the strain when the stress is diminishing and vice versa. Incidentally, the irrecoverable strain which remains after the final removal of all loads is greater for a declining stress history than for an increasing stress history as would be expected from the changing properties of concrete with increasing age. In all cases of ultimate zero stress, the effective modulus method gives an erroneous result since it predicts complete recovery.

Curves computed by the rate of creep method show an agreement with experimental results which is perhaps surprising in view of the theoretical deficiencies of the method. The assumption of an invariable elastic modulus

to estimate increments of elastic strain regardless of the age has sometimes improved and sometimes impaired this agreement. It is noticeable that the method cannot predict a creep recovery following a reduction of stress, so that the curvature in the creep-time curve is sometimes wrong, e.g. between 28 and 63 days in Fig. 11. The shape predicted on complete removal of the load, i.e. a horizontal straight line, is always wrong. The diagrams confirm the theoretical deduction that the rate of creep method overestimates creep under a declining stress. Despite these deficiencies, the correct order of magnitude of strains under widely varying stresses is predicted by this method.

The strain-time curves computed by superposition show a fair agreement with experimental results although they are not markedly superior to those computed by the rate of creep method. The diagrams do, however, have the correct general shape and this suggests that, with the larger creep values experienced under normal conditions, the method of superposition would show a superiority over other methods which is not evident in these tests.

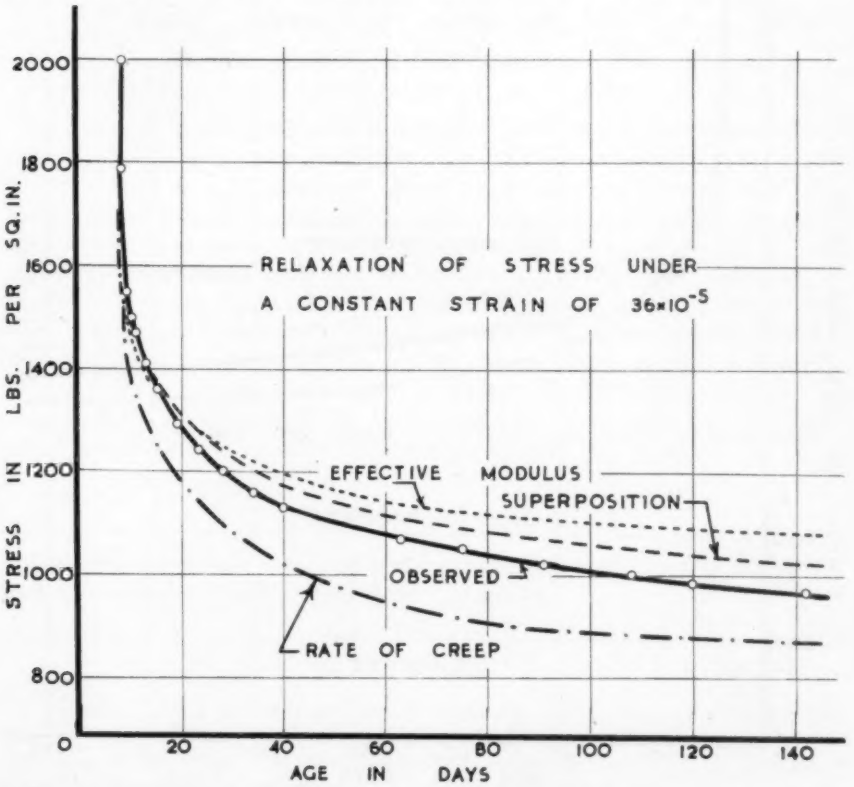

Fig. 12—Relaxation of stress under constant strain. The initial stress of 2000 psi caused an immediate strain of 36×10^{-5} which was thereafter held constant for 140 days

Relaxation of stress under constant strain

The 50 percent reduction in stress in 4 months should be noted despite the small creep of this concrete. With a lower humidity, a weaker concrete and no compensation of shrinkage the entire initial stress would quickly be lost. This has been verified in other specimens of ordinary grade concrete which lost about 80 percent of the initial stress in only 14 days when maintained at constant strain in the atmosphere of the general laboratory. This kind of result emphasizes the importance of creep and shrinkage in the prestressing of continuous pavements.

Considering the computed curves in Fig. 12 and 13, all three methods predict the relaxation of stress fairly well, superposition showing the closest

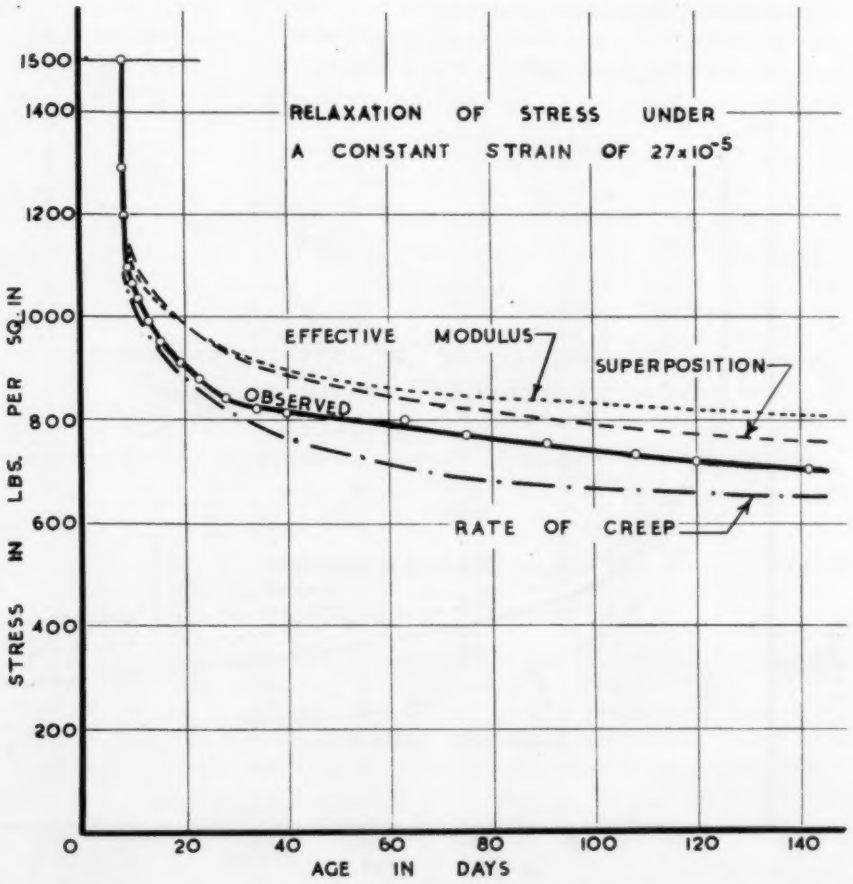

Fig. 13—Relaxation of stress under a constant strain. The initial stress of 1500 psi caused an immediate strain of 27 $\times$ 10^{-5} which was thereafter held constant for 140 days

agreement with experimental results. The effective modulus and rate of creep methods overestimate and underestimate respectively the residual stress; i.e. they respectively underestimate and overestimate the creep under declining stress, thus confirming again the sense of the errors to be expected with these methods. Magnitude of error is about the same in both cases.

CONCLUSIONS

The calculated loss of stress in a typical pretensioned concrete member is practically the same whether it is computed by the effective modulus or by the rate of creep method. In the relaxation tests reported here, a stress loss of 50 percent was predicted with almost equal success by both methods, the true result lying between the predictions. Therefore in structural members in which there is a gradual "natural" decline in concrete stress, e.g. a reinforced column under sustained load or a pretensioned element, the effective modulus can be used with confidence. Superposition will give the best result but one not very different from the approximate result. The rate of creep method has no superiority over the effective modulus method which is therefore greatly to be preferred because of its extreme simplicity and convenience.

On the other hand, if concrete is subjected to abrupt or severe changes of stress, the effective modulus method is subject to serious errors and one of the other methods is to be preferred. From more than one point of view, the method of superposition is superior, but it requires the assembly of much experimental data and is rather laborious to apply. For important problems however, which justify the effort in the laboratory and office, this method would be the choice.[6] On the other hand these tests have shown that from a single creep curve and a single value of the elastic modulus the strain can be predicted with reasonable success by the rate of creep method for widely varying regimes of stress. Despite its theoretical deficiencies the rate of creep method has a considerable utility particularly in cases for which the rather extensive data required for superposition are not available.

ACKNOWLEDGMENT

This paper describes a part of a wider investigation of creep in which the author is assisted, in particular, by S. S. Dhondy.

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Discussion of this paper should reach ACI headquarters in triplicate by June 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

Ultimate Resisting Moment of Beams with Compression Reinforcement*

By EUGENE GUILLARD†

SYNOPSIS

A change in basic assumptions is proposed for ultimate strength design of beams with compression reinforcement. It is shown that the present method is inconsistent for shallow beams and slabs with large percentages of compression steel. A revised form of Eq. (A3) in Section A606 of the ACI Building Code (ACI 318-56) is suggested which permits incorporation of the change in assumptions.

INTRODUCTION

Some doubt has been raised as to the range of applicability of Eq. (A3), Section A606 of the ACI Building Code (ACI 318-56). The uncertainty resulted from a comparison of slabs designed with and without compression steel. The ultimate resisting moment of a slab with tension steel only was computed by Eq. (A1), Section A605. This moment was compared with the ultimate moment, computed by Eq. (A3), of an identical slab having the same tension steel area, but having in addition an equal area of compression steel. The surprising result was that the ultimate moment of the slab had been reduced by the addition of compression steel.

While it is conceivable that the addition of compression steel will not increase the ultimate moment that a beam can resist under some circumstances, it is difficult to believe that the ultimate moment will be reduced. If one adheres to the philosophy that each material will sustain the maximum force that it is capable of resisting, then a reduction in resisting moment is impossible. The inclination is to believe that the assumptions upon which Eq. (A3) is based are not valid for the case cited.

REVIEW OF BASIC ASSUMPTIONS

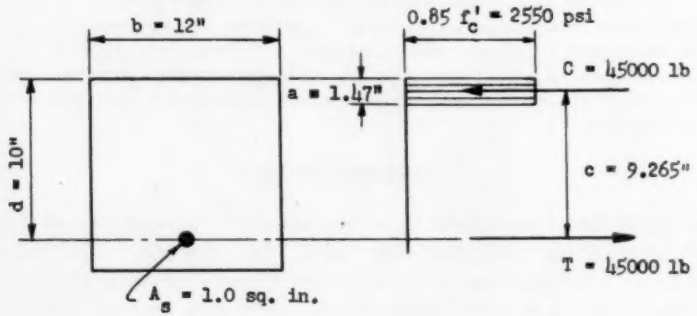
Based on the assumptions‡ underlying Eq. (A1) and (A3) the stress distributions for both slabs are as shown in Fig. 1. Obviously in this case the addition of compression steel area equal to the tension steel area will reduce the ultimate resisting moment of the section. The apparent reduction in

*Received by the Institute May 29, 1957. Title No. 54-42 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Research Engineer, Structural Analysis Section, Armour Research Foundation, Illinois Institute of Technology, Chicago, Ill.

‡Whitney, Charles S., "Plastic Theory of Reinforced Concrete Design," *Transactions*, ASCE, V. 107, Paper No. 2133, 1942.

ultimate moment is a result of the assumption that the compression steel is stressed to the yield point and that the balance of the compression (if any) is carried by the concrete. In this case the assumptions lead to a reduction of the moment arm of the resisting couple. Would it not be more logical to assume that the concrete is capable of resisting the same forces as before the compression steel was added? Is it not true in this case that the top steel or so-called compression steel is ineffective because it lies below the assumed stress block at ultimate load?



$f_c' = 3000$ psi

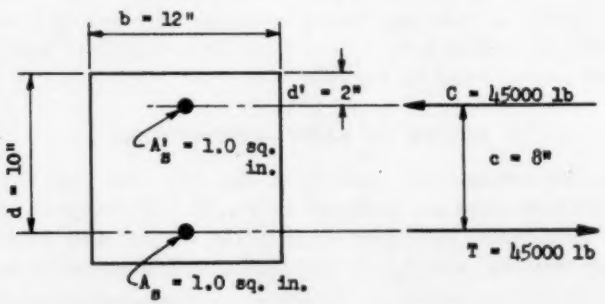
$f_y = 45{,}000$ psi

$C = T = 1.0 \times 45{,}000 = 45{,}000$ lb

$$a = \frac{45{,}000}{2550 \times 12} = 1.47 \text{ in.}$$

$$c = 10.0 - \frac{1.47}{2} = 9.265 \text{ in.}$$

$M_u = 45{,}000 \times 9.265 = 417{,}000$ in.-lb



$M_u = 45{,}000 \times 8 = 360{,}000$ in.-lb

Fig. 1—Stress distribution in beams at ultimate load; at top is beam with tensile reinforcement only. Beam at the bottom has both compressive and tensile reinforcement

Maximum ultimate resisting moment

In an attempt to obtain supporting data for affirmative answers to the questions posed above, Eq. (A3) was rewritten in a form similar to that originally presented.* Thus, Eq. (A3) in the form

$$M_u = (A_s - A_s')\, f_y\, d\,[1 - 0.59\,(p - p')\, f_y/f_c'] + A_s'\, f_y\,(d - d') \quad \ldots\ldots (1)$$

becomes

$$\frac{M_u}{f_y\, b\, d^2} = \frac{(d - d')}{d}\, p' + (p - p')\left[1 - (p - p')\,\frac{m}{2}\right] \quad \ldots\ldots (2)$$

where $m = f_y/0.85\, f_c'$

Substituting $\alpha\, p$ for p' and rearranging this may be written as

$$\frac{M_u}{f_y\, b\, d^2} = p\left(1 - \alpha\,\frac{d'}{d}\right) - \frac{p^2\, m}{2}\,(1 - \alpha)^2 \quad \ldots\ldots (3)$$

Differentiating Eq. (3) with respect to α, and setting the derivative equal to zero results in

$$\alpha = 1 - \frac{d'/d}{pm} \quad \ldots\ldots (4)$$

Since α is equal to p'/p, the maximum ultimate resisting moment M_u of a beam with compression steel is obtained when

$$\frac{p'}{p} = 1 - \frac{d'/d}{pm} \quad \ldots\ldots (5)$$

The maximum value of M_u occurs when the percentage of compression steel is just sufficient to raise the lower edge of the rectangular stress block on the concrete to the centroid of the compression steel area (see Fig. 2). If more compression steel is added the stress block becomes smaller and the top steel falls below the stress block resulting in a lower computed resisting moment by the present formula.

Fig. 3 is a plot of $M_u/(f_y\, bd^2)$ and p'/p for a few values of p and d'/d based on an assumed value for m. For Curve A, which is representative of the case originally cited, the stress block without compression steel is above the level at which the compression steel is placed, and consequently the ultimate resisting moment as computed by Eq. (A3) is lower for all percentages of compression steel than for the beam with only tension steel.

*See Whitney paper cited on p. 759.

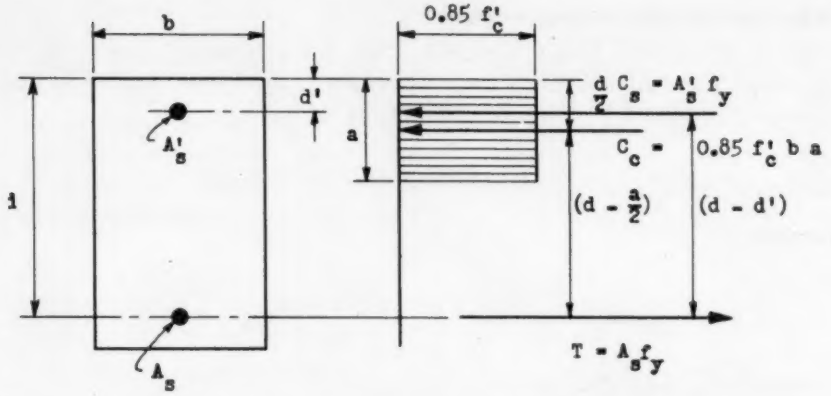


Fig. 2—Stress distribution for maximum ultimate moment for beams with compressive reinforcement. Taking moments about the compression steel

$$M = A_s f_y (d - d') - 0.85 f_c' b a \left(\frac{a}{2} - d'\right)$$

$$\frac{dM}{da} = -0.85 f_c' b a + 0.85 f_c' b d'$$

For maximum moment

$$-0.85 f_c' b a + 0.85 f_c' b d' = 0$$

$$a = d'$$

PROPOSED CHANGE IN ASSUMPTIONS

The writer believes that for the curves of Fig. 3 the portion to the right of the point of maximum has no meaning and that, in fact, the curves should be extended as shown by the dashed line on Curve *B*. This will make the curves agree with the assumption that the concrete is stressed to its ultimate, and that the stress in the compression steel is reduced below the yield stress because there is an excess of steel. (In the case originally cited the stress in the compression steel will be zero.)

Revised formula for ultimate moment

If the above arguments are accepted, then Eq. (A3) of the Code can be presented in a form which will not only yield the ultimate moment, but also give the optimum percentage of compression steel. It is suggested that Eq. (A3) be written

$$M_u = f_y b d^2 \left[p \left(1 - \alpha \frac{d'}{d}\right) - \frac{p^2 m}{2} (1 - \alpha)^2 \right] \quad \ldots\ldots\ldots\ldots (6)$$

where

$$\alpha = 1 - \frac{d'/d}{pm} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The equation should be used in the following manner to determine M_u for a given beam section:

Compute p'/p and compare with the value of α from Eq. (7). If p'/p is smaller than α, use p'/p for α in Eq. (6). If p'/p is larger than α, use α directly in Eq. (6). If α is negative (the compression steel is unstressed), use α equal to zero in Eq. (6).

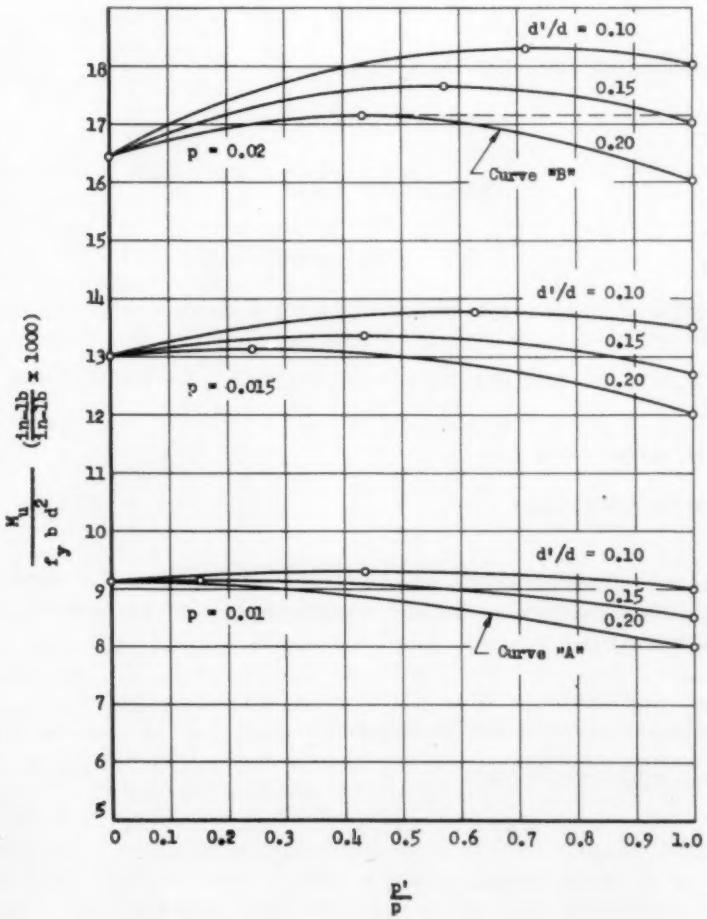


Fig. 3—Ultimate moments of beams with compression steel based on Eq. (A3) of the ACI Code

$f_c' = 3000$ psi; $f_y = 45{,}000$ psi; $m = f_y/0.85\,f_c' = 17.6$

Illustrative example

Given a concrete beam having the following dimensions and properties, find the ultimate resisting moment.

b	= 12 in.	f_c'	= 3000 psi
d	= 15 in.	f_y	= 45,000 psi
d'	= 3 in.		
A_s	= 2.70 sq in.	p	= 0.015
A_s'	= 2.16 sq in.	p'	= 0.012

By present Code (ACI 318-56) Sec. A606

$$\text{Check: } (p - p') \leqq 0.40\, f_c'/f_y$$

$$(0.015 - 0.012) \leqq 0.40 \times \frac{3}{45}$$

$$0.003 < 0.0267 \qquad \underline{\text{OK}}$$

$$M_u = (A_s - A_s')\, f_y d\, [1 - 0.59\, (p - p')\, f_y/f_c'] + A_s'\, f_y\, (d - d') \quad \text{.....(A3)}$$

$$M_u = (2.70 - 2.16)\, 45 \times 15 \left[1 - 0.59 \times 0.003 \times \frac{45}{3}\right] + 2.16 \times 45 \times 12$$

$$M_u = 355 + 1165$$

$$M_u = \underline{1520 \text{ in.-kips}}$$

Note that this is actually a lower resisting moment than that obtained by omitting the compression steel and computing the ultimate moment by Eq. (A1) of Sec. A605.

By present Code (ACI 318-56) Sec. A605

Tensile reinforcement only

$$q = p\, f_y/f_c' = 0.015 \times \frac{45}{3}$$

$$q = 0.225$$

$$M_u = bd^2\, f_c'\, q\, (1 - 0.59\, q) \quad \text{.....(A1)}$$

$$M_u = 12 \times 15^2 \times 3 \times 0.225\, (1 - 0.59 \times 0.225)$$

$$M_u = 1823\, (1 - 0.133)$$

$$M_u = \underline{1580 \text{ in.-kips}}$$

By proposed formula

$$\frac{p'}{p} = \frac{0.012}{0.015} = 0.80 \qquad m = \frac{45}{0.85 \times 3} = 17.6$$

$$\frac{d'}{d} = \frac{3}{15} = 0.20 \qquad \alpha = 1 - \frac{d'/d}{pm} = 1 - \frac{0.20}{0.015 \times 17.6}$$

$$\alpha = 0.243$$

$\frac{p'}{p} > \alpha$; therefore use $\alpha = 0.243$ in Eq. (6)

$$M_u = f_y bd^2 \left[p \left(1 - \alpha \frac{d'}{d} \right) - \frac{p^2 m}{2} (1 - \alpha)^2 \right] \qquad (6)$$

$$M_u = 45 \times 12 \times 15^2 \left[0.015 \, (1 - 0.243 \times 0.20) - \frac{0.015^2 \times 17.6}{2} (1 - 0.243)^2 \right]$$

$$M_u = 121{,}500 \, [0.0143 - 0.0011] \qquad M_u = \underline{1600 \text{ in.-kips}}$$

In this case there is an excessive amount of compression steel which may be unusual but not necessarily unrealistic. The optimum percentage of compression steel is

$$p' = \alpha \, p \qquad \text{or} \qquad A_s' = 0.0036 \times 12 \times 15$$

$$p' = 0.243 \times 0.015$$

$$p' = 0.0036 \qquad A_s' = 0.65 \text{ sq. in.}$$

CONCLUDING REMARKS

The objection may be raised that the equation in this proposed form does not permit a correction for the concrete area displaced by the compressive reinforcement. It is believed that this is not a serious shortcoming of the equation, since many designers consider this a trivial refinement. Another possible objection to the equation is that the yield strength of both tensile and compressive reinforcement must be the same. However, different yield strengths for each could be taken into consideration by a correction factor for p' based on the relative yield strengths.

In conclusion it should be stated that the suggested revision of Eq. (A3) is only important for a small number of problems encountered by the design engineer. Nevertheless, it is a refinement of the ACI Building Code that can be easily applied, and agrees with the general concepts of ultimate strength design.

Discussion of this paper should reach ACI headquarters in triplicate by June 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

First Slip-Formed Apartment Building in the United States*

By JOHN H. DOGGETT†

SYNOPSIS

Upper eight stories of ten-story apartment building were slip formed in 115 hr of around-the-clock operations. Structural framing, building of the forms, and construction practices are described, citing advantages peculiar to the method.

INTRODUCTION

The upper eight stories of the ten-story Holiday Towers, Memphis, Tenn., were constructed by slip forming. The decision to adopt this method to construct an apartment building came about as the result of investigations made by the architect, contractor, and owner. The virtual elimination of scaffolding and repeated handling of wall forms indicated both speed and economy in slip-form construction. The over-all plan was to cast all exterior and major interior walls simultaneously, and continuously, to roof deck level. With careful planning and an experienced contractor, all felt reasonably certain it was feasible.

Stores, parking facilities, and column arrangement in the basement and first two stories rendered conventional forming methods more practical for these floors, which are 90 x 148 ft in plan. The upper eight slip-formed stories, 70 x 144 ft, contain 160 efficiency apartments, ten on either side of a central corridor on each floor.

FOUNDATIONS

The lower cast-in-place stories required as much clear space as possible for parking automobiles. It was decided, therefore, to provide a single row of columns in the basement (Fig. 1) and first story along the longitudinal center line of the ten-story portion of the building. Third-story crosswalls were heavily reinforced and set on a shallow girder to support the load from the walls above and transmit it to exterior walls and a center column.

The center corridor in the upper eight stories made a continuous wall span impossible. Therefore, the corridor ends of the crosswalls are supported by

*Adapted from a paper presented at ACI regional meeting, Atlanta, Ga., Nov. 1, 1955. Title No. 54-43 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than June 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

†Architect, Memphis, Tenn.

Fig. 1—General view of foundation work showing center-line columns designed for million-lb load

large V-shaped concrete members on the second floor which bear on the single row of columns on the first floor.

Since the load on each of these center columns is approximately 1,000,000 lb, it was necessary to use combined footings to support these columns. Vertical stirrups were needed in these footings to resist diagonal tensile stresses. All other footings, excepting one which carried the corner column, were conventional rectangular footings.

FRAMING THE LOWER FLOORS

The basement and first floor were beam and slab construction. On the second floor it was necessary to construct the V-shaped (Fig. 2 and 3) members with legs 24 in. square. They extended from the tops of the first floor center columns to column cap beams at the ceilings of the second story. These caps were wide portions of the floor made deeper to reduce shearing stresses. Tops of the legs of the V-shaped members were 7 ft 4 in. apart at a point 10 ft above the second floor slab.

Heavy column steel was required in the cast-in-place portion; center columns are 32 in. square and contain 22 #11 bars.

WALL DETAILS

Continuous horizontal keyways, 1⅝ in. deep and 4 in. high, were formed on the inside face of the north and south exterior walls. Dowels at 6-in. centers, bent to allow passage of forms, projected from near the top of the keyways and were later used for tying in floor reinforcing. This keyway and dowel detail was also used in the stairwell areas of the north and south walls. Little patching was required later.

Each room on the east and west exterior walls has two short keyways, 1⅝ in. deep, 4 in. high, and 2 ft long, with projecting dowels. Tops of keyways are located at the tops of floor slabs. Openings, 4 in. high by 3 ft long, were boxed out in the corridor walls at each floor level. Thus when floors were placed, it was possible to carry reinforcement through these openings. Keyways 4 in. high by 1 ft 6 in. to 4 ft long were formed in the crosswalls at the floor level. This keying system resulted in one vast, well integrated slab as large as the building in area.

Doorways were provided in all crosswalls by 4 x 7 ft wood frames. These openings facilitated room-to-room passage during floor construction. Although these doorways were later blocked with concrete masonry and plaster, they could be easily reopened to provide multiroom apartments.

Vertical wall reinforcement splices were staggered to avoid peak and valley work loads on the men placing the reinforcement. Well detailed placing drawings were prepared by the structural engineer for all walls.

CONSTRUCTION OF SLIP FORMS, JACKING

Slip forms were carefully laid out and constructed immediately after the third floor slab had been cast and cured. Above this slab, the design of the building was based on all cast-in-place walls, elevator shafts, and boiler stack being cast in a single operation. Both outer and inner walls are 7½ in. thick, and pilasters are 13 in. thick.

The completed inner forms were 13 ft 6½ in. x 30 ft 9 in. in plan, 3 ft 6 in. high. Vertical 1 x 4 tongue-and-groove boards were attached to double 2 x 8

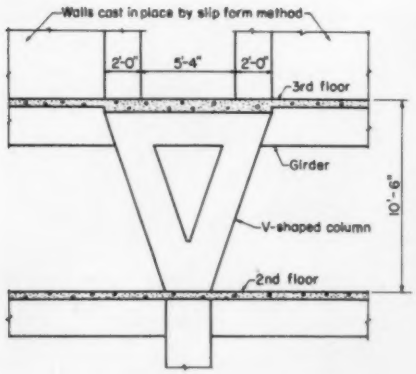


Fig. 2—V-shaped column between second and third floors effected transition between column system and bearing walls above

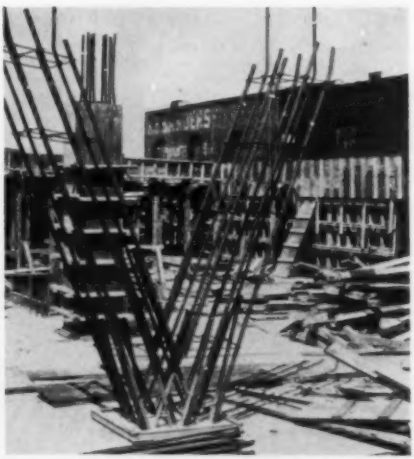


Fig. 3—Reinforcement for special V-shaped column used between second and third floors

Fig. 4—Slip forms under construction on the third-floor slab. Carpenter attaches vertical tongue-and-groove boards to double 2 x 8 wales. Slight draft to form was obtained by nailing a strip ¼-in. thick on the edge of upper wale

wales to fabricate the inside form (Fig. 4). Wales in the outer wall forms were notched to provide additional thickness for pilasters between windows. The forms, canted approximately ⅛ in. to facilitate movement, were given one treatment of oil just prior to start of concreting.

Working platforms between wall forms were used afterward as forms for casting the roof slab. Metal yokes aligned the inner and outer slip forms and maintained desired wall thickness.

Jacks and jack rods

A total of 210 interconnected hydraulic jacks attached to the metal yokes (Fig. 5) uniformly lifted the slip forms and working platform on 1-in. jack rods made continuous by threaded unions staggered in the vertical plane. After the walls had been completed, the jack rods were pulled. Jack rods were encased in a close-fitting pipe sleeve that moved with the yoke assembly. Rods were pulled by hand. Horizontal alignment of yokes and forms was maintained by the use of instrument and on the basis of common water level in hoses leading to each yoke assembly.

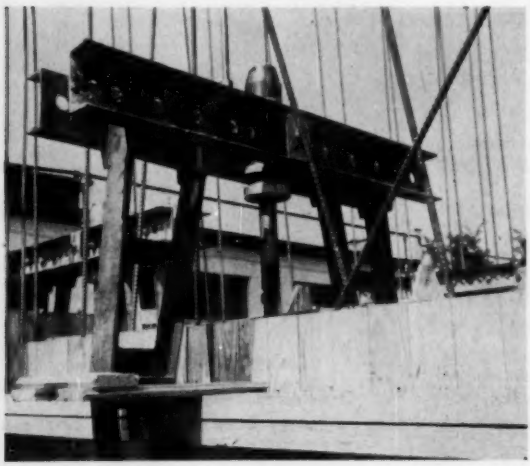


Fig. 5—Metal yokes aligned outside and inside forms, regulated wall thickness, and mounted jacking equipment

WALL CONSTRUCTION

The upper eight slip-formed stories, which were concreted continuously, required the placing of about 2100 cu yd of concrete in approximately 115 hr. This concreting schedule required three crews of men on 8-hr shifts. Concrete was transported by buggy from elevator to forms. It was placed in 6-in. lifts and consolidated by hand spading. Ready-mixed concrete was supplied for the entire project on an around-the-clock schedule. Most of the concrete was designed for 3000 psi strength at 28 days; it had a 5-in. slump and contained 5½ sacks Type I cement per cu yd.

Speed of the slip forms, which were raised in 1-in. increments, ranged between 6 to 12 in. per hr. Due to coolness and humidity, better progress was made at night.

Careful planning by the contractor's general superintendent greatly facilitated uninterrupted operations. All reinforcement had been delivered, stored, and fully labeled before construction began. Frames for openings, which had been built in a shop at the lower level, were readily available.

The placing and concentration of steel called for in the third-story walls, due to their use as deep girders, was complicated at first; but by the time the walls had reached the fourth floor, the workmen had developed an operational routine that proved satisfactory for the remaining stories. Fig. 6 shows progress on the fourth day.

Finishers working on a scaffold suspended from the form patched the concrete where necessary and gave the exterior surface a wood float finish to eliminate any board markings. A perforated garden hose slung beneath the forms provided an adequate water spray to assure proper curing.

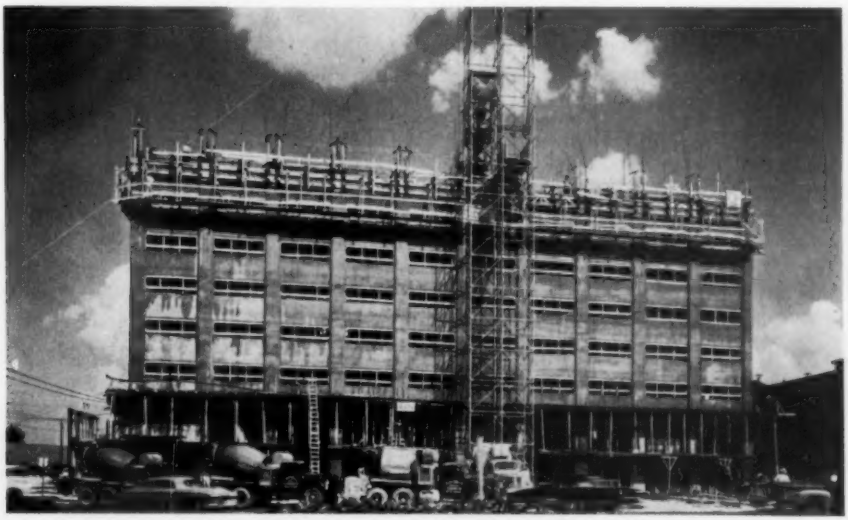


Fig. 6—By the fourth day walls were nearing completion

COMPLETION OF STRUCTURE

A week was allowed for the walls to cure, and then the roof slab was placed using the working platform as a form. After the roof concrete had reached a predetermined strength, a new form was raised from the third to the tenth floor. This form rested on two wide-flange beams which in turn were supported by three 1¼-in. rods per beam from the roof slab.

After the tenth floor concrete had been placed and acquired sufficient strength, the beams and forms were lowered to the ninth floor and secured by the rods to the tenth floor. This routine was continued to the seventh floor. During this time the fourth, fifth, and sixth floor slabs were placed by conventional methods to hasten completion of entire project.

As floors were completed, plumbing and mechanical installation proceeded. After window sills had been cast in place, the exterior walls were painted a light tan. Three coats of acoustical plaster were sprayed directly on the ceilings to ¼- to ½-in. thickness. Walls were given a sand float finish.

The building was turned over to the owners in February, 1956. It would have been completed in the originally scheduled 10 months, had it not been for a 2-month delay caused by a strike at the plant of supplier of the air conditioning equipment. The contractor estimates that costs for the upper eight stories were 10 percent less than if conventional forming had been used and that from 3 to 5 months were saved in construction time.

CONCLUSIONS

Final review of costs and methods indicate:

1. Forming and casting floor slabs, except roof slab, in conventional manner would be more economical, due to difficulty in rigging and leveling forms.

2. Continuous concrete placing and movement of forms causes increased labor costs for overtime. Consideration has since been given to placing and "slipping" continuously three floors, stopping, forming floor slabs, then slipping three more floors, and so on to completion.

3. End results were satisfactory to all parties concerned, considering the difficulties and circumstances peculiar to this job.

ACKNOWLEDGMENTS

John H. Doggett, Memphis, was the architect; structural engineers were S. S. Kenworthy and Associates, Memphis. Southern Builders Inc. was the general contractor, and ready-mixed concrete was supplied by Schevenell Co., Memphis.

Discussion of this paper should reach ACI headquarters in triplicate by June 1, 1958, for publication in the Part 2, September 1958 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Various methods of calculation for prestressed concrete bridges with leg-frame supports (Différentes méthodes de calculs des ponts á béquilles en béton précontraint)

R. SCHERE, *Travaux* (Paris), No. 270, Apr. 1957, pp. 213-216

Describes methods of analysis with inclined supports. Two "not unusual" methods of calculation are described briefly. A third method described in detail is applicable to structures in the form of an arch which consist of a deck of variable moment of inertia and a triangulated leg-frame at each end.

Load factors for prestressed concrete bridges

T. Y. LIN, *Proceedings*, ASCE, V. 83, No. ST 4, July 1957, pp. 1315-1–1315-18

AUTHOR'S SUMMARY

The use of load factors in ultimate design for prestressed concrete bridges is compared to the elastic theory and allowable stress method. Variables affecting the load factor of bridge sections are analyzed, indicating the wide divergence when designed on the basis of no tension in concrete. In addition to the load factors for flexural strength of simple and continuous spans, factors for strength at transfer, for shear, and for tension and compression members are discussed. The significance of load distribution, load repetition, and the meaning of yield point and usable strengths are brought out. Load factors for total load are listed, as well as for dead, live, and wind loads.

Insulated forms for winter construction of bridges

K. R. SCURR, *Bulletin* No. 162, Highway Research Board, 1957, pp. 13-19

Describes the use of modern insulation on the forms for bridge structures in South Dakota. Costs, temperature records, and details of the construction are illustrated. Details showing the application of insulation to concrete box culverts, concrete slab bridges with steel girders, and concrete bridge piers and walls are shown.

Construction Techniques

Status of formwork and scaffolding technique (Stand der Schalungs- und Rüstungstechnik)

A. GATTNAR, *Die Bauzeitung* (Stuttgart), V. 62, No. 5, May 1957, pp. 220-222, also p. XXII

Reviewed by ARON L. MIRSKY

Brief survey of current European techniques and their development over the last decade.

Resistance of vacuum-treated concrete to frost and wear (in Swedish)

JOHN NYGARDS, *Nordisk Betong*, V. 1, No. 3, 1957, pp. 251-262

Reviewed by MARGARET CORBIN

Sets forth advantages of vacuum treatment in placing concrete. A wetter consistency can be used and makes consolidation easier. Water accumulating on the surface of the concrete is extracted by vacuum leaving the surface dry and doing away with later "bleeding" phenomena. A maximum percentage of coarse aggregate can be used, as compaction is facilitated by the wetter mix. Tests conducted in Sweden show that

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For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

frost resistance is improved and wear resistance, which is dependent on the evenness of the surface, is also benefited. Test results are given.

Dams

Winter concreting in Ikawa Dam (in Japanese)

HIDEO WAKU and TETSUO NIWA, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 42, No. 10, Oct. 1957, pp. 1-6

Reviewed by KIYOSHI OKADA

Describes equipment and installations for winter concreting and the thermal efficiency in Ikawa Dam, a hollow gravity dam constructed in central Japan, where minimum temperature is about − 10 C. Batching plants and the many conveyors are heated by steam boilers, electric heaters and/or infra-red lamps. Thermal efficiency of 18 percent is obtained in batching plant in mid-winter night.

Some experiences regarding design and construction of a gravity dam in Anatolia (Einige Erfahrungen über Entwurf und Bau einer Schwergewichtsstaumauer in Anatolien)

W. JURECKA, *Oesterreichische Bauzeitschrift* (Vienna), V. 12, No. 7-8, July-Aug. 1957, pp. 149-156

Reviewed by H. H. WERNER

A description of this project regarding general conditions as to: layout, site, materials, and manpower. The selected type of dam and its construction are given in detail, together with plant layout, construction methods, use of precast form-slabs, and construction joints.

Design

Explanatory handbook on the British Standard Code of Practice for Reinforced Concrete, No. 114, 1957

W. L. SCOTT, W. H. GLANVILLE, and F. G. THOMAS, Concrete Publications Ltd. (London), 3rd Edition, 1957, 155 pp., $3

A text for practicing engineers furnishing an authoritative interpretation of the British Standard Code of Practice. The code itself is reproduced in its entirety, clause by clause, with design tables, figures, explanations, and examples to illustrate each clause and requirement. The handbook has been revised to include charts for design by the ultimate strength method in addition to those for use with the elastic theory.

Some notes on the ultimate bending strength of prestressed continuous beams (Quelques aspects de la résistance á la rupture en flexion des poutres continues précontraintes)

JEAN MULLER, *Travaux* (Paris), No. 272, June 1957, pp. 315-331

The plastic hinge theory is applicable in most cases to prestressed concrete continuous beams. Recourse may be had to the equations and formulas given in the body of the paper to predict within good accuracy the ultimate moment of a prestressed concrete section with bonded reinforcement.

While several tests confirm the validity of the plastic hinge theory, certain particular cases proved that complete redistribution of moments as assumed by this method was not quite obtained at the time of failure.

New method of analysis for slabs with multiple openings, exemplified by Vierendeel-type girders (Ein neues Berechnungsverfahren für gegliederte Scheiben, dargestellt am Beispiel des Vierendeelträgers)

HUBERT BECK, *Der Bauingenieur* (Berlin), V. 31, No. 12, Dec. 1956, pp. 436-443

Reviewed by ARON L. MIRSKY

Problem arose in analyzing a solid multistory end wall with multiple rectangular openings. As a simpler structure of same type, serving as a preliminary to solution of main problem, author investigates Vierendeel-type girder. For analysis, this is replaced by a single-celled rigid frame (top, bottom, two end members) with the intermediate posts replaced by lamellas of length dx. Formulas are derived for several special cases (of support, loading); as a numerical example, nine simple Vierendeel girders are analyzed by this method and by usual frame-analysis methods and results are compared. Extension and application to multistory shear walls will be given in a later paper.

Comparative study of stress in shell roofs (in Italian)

ELIO GIANGRECO, *Giornale del Genio Civile* (Rome), V. 94, No. 11-12, Nov.-Dec. 1956, pp. 739-748

Reviewed by JOHN I. PRELEZ

Presents comparative study of reactions of different designs and shapes of shell roofs with different reinforcing coefficients.

Discussion of a report by ACI Committee 214:

Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)*

By CELSO A. CARBONELL, HARRY H. MITCHELL, and COMMITTEE

By CELSO A. CARBONELL†

There are a few matters on this interesting subject that I would like to call to the attention of the committee hoping to help clarify the recommendations. On p. 2 the report says, "To place too much reliance on too few tests may result in erroneous conclusions." Agreed. However, on p. 12 it is stated that in a centralized project and "under adequate control of all operations, it is unnecessary to test the concrete at the point of placement." This is the only way of knowing that the concrete is arriving as expected at the point of placement. Also, any necessary adjustments could be recommended for the next batches after one is proven unsatisfactory. On p. 13 the committee says, "Each different type of concrete placed during any one day will be represented by two standard 6 x 12-in. cylinders to be tested at 28 days." Perhaps it would be better to increase to three the number of cylinders in order to really make it a test.

Also on p. 13 is the following statement, "The popular practice of purchasing concrete from a separate producer and contracting the testing work separately complicates concrete testing and control." If a reliable laboratory is engaged to do the testing there is no problem whatsoever either in control or in testing, and besides it proves to the customer that the producer is so confident in his product that he will let others verify his claims. Besides, we must encourage the establishment and development of more good laboratories that will mean more research for better and more economical production of concrete for all type of work.

I like the statement on p. 14 that tests on job-cured specimens "should not be confused with, nor replace, standard control tests." This is of utmost importance. The only real tests are those conducted in accordance with standard procedures; all others should be considered of limited referential value. This must be strongly emphasized for there is the tendency to consider that job-cured specimens represent the concrete as it actually is and are consequently superior for testing. We all know the danger of such implication.

*ACI JOURNAL, July 1957, *Proc.* V. 54, p. 1. Disc. 54-1 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Executive Vice-President and General Manager, Sika Panama, S.A., Panama City, Panama.

On p. 15 another important point is dealt with—rejection of doubtful test specimens. I believe that not a single test specimen should be rejected unless there is definite proof that it does not represent the conditions of the concrete for reasons well known.

I have studied the statement on p. 3, "Use of admixtures presents additional problems in maintaining uniformity of strength since each admixture adds another variable to concrete." I strongly disagree with this statement; admixtures are chosen by the concrete engineer after a careful study of the concrete, precisely to avoid or minimize the disadvantages found in certain cases in the ingredients of concrete. Admixtures are tools in the hand of the concrete engineer to give shape to his final work—good concrete. How could they be considered as additional hazards when they are choosen after knowing exactly what they will do? Admixtures in the hands of the inexperienced are as harmful as a hammer in the hands of a baby, but admixtures of recognized prestige in the hands of a concrete engineer are not additional hazards, but useful tools.

By HARRY H. MITCHELL*

Committee 214 is to be congratulated for this excellent report. We have been using these statistical methods to analyze our concrete for the past 2 years, and are encouraging architects to write their specifications around Table 5, p. 15.

However, it is this writer's opinion that Table 5 should be changed. Under the heading of "structural concrete," the minimum strength requirement for the average of three consecutive tests under ultimate strength design is $1.00 f_c'$; for four, $1.04 f_c'$; five, $1.07 f_c'$, and for six, $1.09 f_c'$. While not likely, it is entirely possible for the average of any number of tests to be above f_c' and yet be below the stated coefficient of f_c'.

This example comes from our files of this year, where $f_c' = 3000$ psi.

Test	*Strength (average of two cylinders)*
C-94	3050 psi
C-95	3060 psi
C-96	3010 psi
C-97	3100 psi
C-98	3200 psi
Average	3084 psi

The required strength for ultimate design by Table 5 would be $1.07 f_c'$ or 3210 psi, and conceivably this example could be classed as a failure; yet not single test fell below the required strength.

It is suggested that four, five, and six tests under ultimate strength, and No. 5 under "general" be left at $1.00 f_c'$, or be left blank, or use Table 5 as given in the proposed recommended practice published in the ACI JOURNAL, Dec. 1956, *Proc.* V. 53, p. 575.

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COMMITTEE CLOSURE

The committee appreciates expressions from those of the concrete industry who have used the committee recommendations in actual practice.

The values shown in Table 5 are theoretical and are based on about the same probability of occurrence of low averages for any given class of concrete under average conditions. The example of five tests given by Mr. Mitchell must be considered a failure of Table 5 requirements because theoretically the probability of a test falling below 3000 psi is greater than one in twenty, although as the example indicates, it is not as great as one in five.

The committee report does not intimate, as Mr. Carbonell suggests that commercial testing laboratories or admixtures are undesirable, but simply points out that if one engineer controls all operations on a project, his job is simplified and there is less need to make control tests both at the mixing plant and at the point of placement.

Since admixtures influence the strength of concrete, it follows that any variation in the admixtures will add to the variations in concrete strength.

Two or even one test per day is not necessarily "too few tests" as Mr. Carbonell suggests, since in one month's time a plant will accumulate what is considered a statistically reliable "large sample."

Discussion of a paper by Herbert A. Sawyer, Jr. and Jack E. Stephens:

Under-Reinforced Concrete Beams Under Long-Term Loads*

By STEFAN SORETZ and AUTHORS

By STEFAN SORETZ†

The authors found that the ultimate curvature and the concrete strain at ultimate load at time of failure of beams under long-term load are about twice as great as in case of the short-term experiment. From the known relations between ultimate curvature, strain at ultimate load, and deflection of the beams one could conclude an equal line of action of the deflections of the beams; that is, the deflections at failure under long-term load are considerably greater than under short-term load.

To obtain information about the influence of a long-term load on the formation of cracks we made extensive experiments, already published.‡ In these experiments deflections were measured too. We think it efficient to compare the fundamental courses of deformation in both cases.

Fig. A shows the dimensions and reinforcement of the test objects. The 28-day strength of the concrete was 3920 psi. The main reinforcement consisted of Torsteel 60 with a permissible stress of 49,700 psi. The additional reinforcement was 6 mm diameter mild steel (St 37). The slabs ("T-beams") were directly loaded at the third-points as shown in Fig. B.

The slabs of Series *a* were tested after 28 days with short-term load (about 12 hr). Those of the Series *b* and *c* were tested after the same time with a gradually increased long-term load; these tests together lasted more than 3 years. Series *b* and *c* differed in that the slabs of Series *b* were placed in a hall heated in winter, where the temperature was not uniform. Those of Series *c* were stored at a steady temperature of 20 ± 5 C. At the end of the long-term experiments the concrete had a cube strength of 5060 psi.

Fig. C shows the Slabs 1 and 2 of the Series *a* and *c* as an example of the temporary course of these experiments. In Fig. D and E one may see the relation between deflection and steel stress, measured at midspan. The steel stress is calculated from given load using $n = 15$. In case of the long-term loads the modification of the deflection in behalf of the load duration is added. During the first long-term load of 210 days all the test objects were free of

*ACI Journal, July 1957, *Proc.* V. 54, p. 21. Disc. 54-2 is a part of copyrighted Journal of the American Concrete Institute, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Engineering Consultant, Tor-Isteg Steel Corp., Luxembourg, Luxembourg.

‡Soretz, Stefan, "Tests on Slabs with Long-Term Load," *Die Bautechnik* (Berlin), V. 33, No. 1, 1956.

cracks. In the second stage of long-term load cracks were present in all cases.

From these test results the following were established:

1. With the long-term load the greatest influence upon the deflection takes place at the first loading increment, during which the cracks are not yet present.

2. In the next long-term loading increment at which the cracks were already present, the influence of the load duration on the deflection was absolutely and relatively smaller. This influence increased slowly with rising load, as long as the steel stress remained in the elastic range. After the elastic limit was exceeded (at a steel stress of about 85,200 psi) the deflection was more and more influenced by time.

3. After a long-term load significantly greater stiffness was ascertained at each following increase of load.

4. The total deflection of the slabs at failure did not show any sure dependence upon the test method—short- or long-term load.

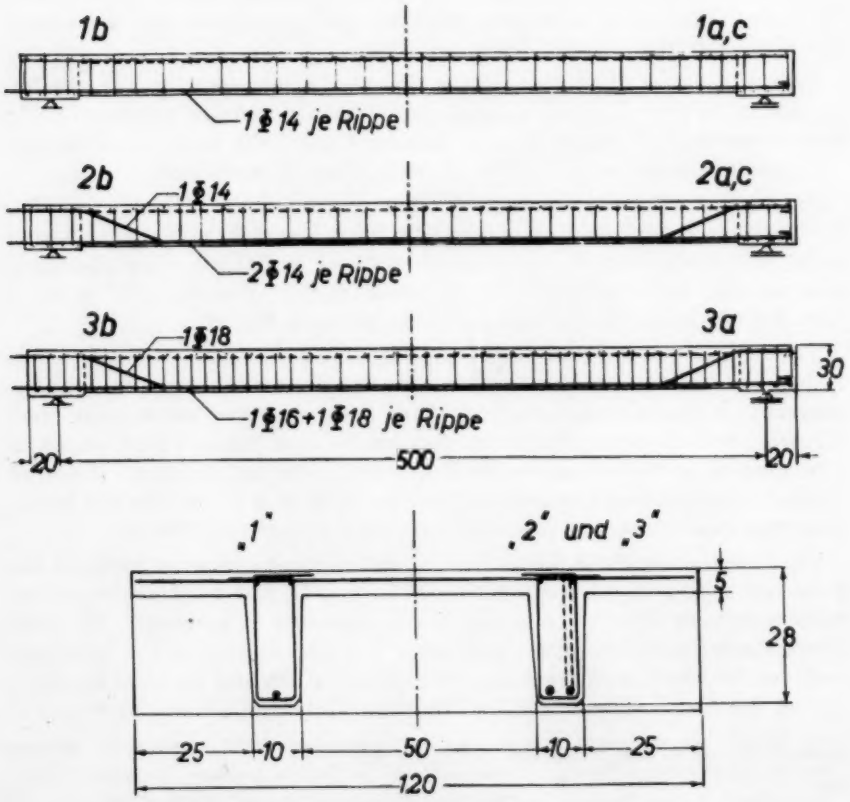
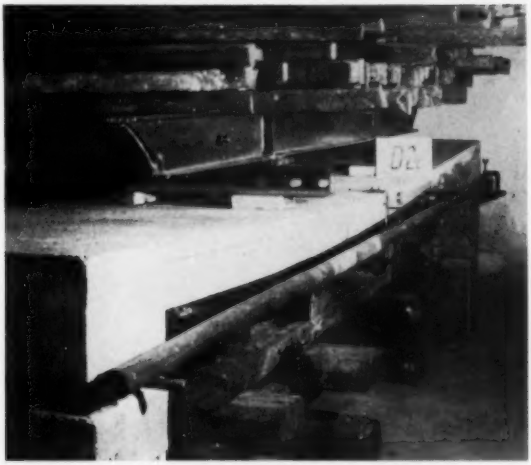


Fig. A—Dimensions and reinforcement of the test object

Fig. B—Loading of the slabs



Fundamentally the results of these experiments differ on some points from those of the authors. There are, however, basic differences too in the shape of the test objects and in the execution of the tests. To compare the test methods, the corresponding curves of Tests II S and L as well as VI S and L from the report of the authors are recorded in Fig. C. Table A compares the various phases of the two test programs.

TABLE A—COMPARISON OF THE TWO TEST PROGRAMS

Sawyer-Stephens	Soretz
Objects tested, rectangular beams	Slabs ("T-beams")
Short-term test at the end of the long-term test	Short-term test in the beginning of the long-term test
Start of the long-term load at two-thirds to three-quarters of ultimate load	Start of the long-term load at 10 to 20 percent of ultimate load
Short-term loads last 1 hr. Long-term loads last 44 to 75 days	10 hr for short-term loads, 1200 days for long-term
Load duration of each degree 4 days State of constancy not yet reached	100 to 335 days until the state of constancy reached
Load increments at long-term tests 2 percent of the ultimate load	10 to 30 percent
Greater stiffness when load increases after long-term load	The same
Influence of the load duration upon the deformation under comparative load intensity is greater here	Here smaller
Deformation at failure after long-term load twice as great as at failure after short-term load	Deformation at failure after long-term load is about the same as after short-term load

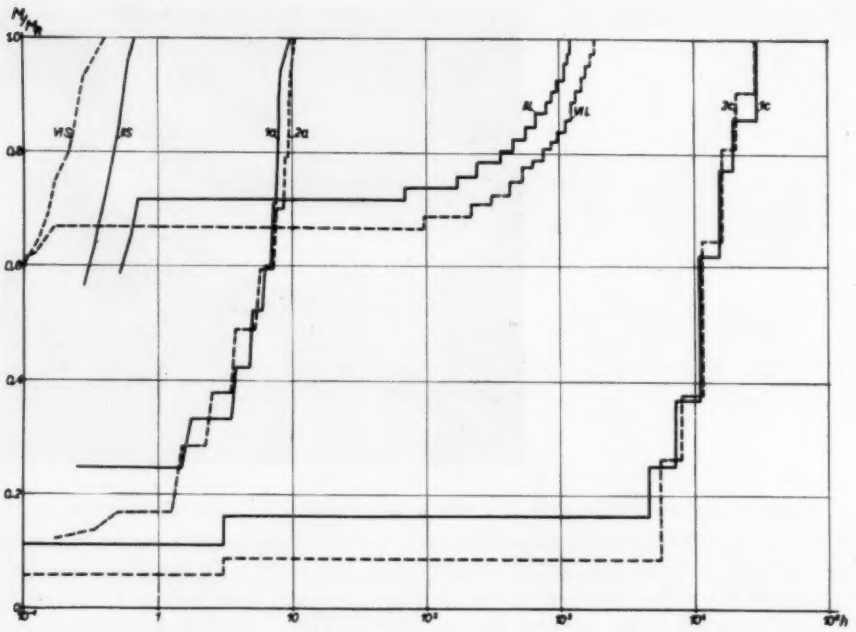


Fig. C—Moment-time curves comparing Soretz' results with those of the authors

Both researches concur quantitatively as well as qualitatively on one point—greater stiffness under increase of load after long-term load. Comparing two final points (Table A) one meets real oppositions, which might be explained in the first instance by the different test methods. In Case II the long-term load starts at 15 percent of the ultimate load and before the appearance of the first cracks; in Case I, however, one starts at 70 percent of the ultimate load, that is under an intensity of load about five times as high. The individual increments of the long-term load in Case II are about ten times as great as in Case I. The total test duration in Case II is 10 to 30 times and the load duration at each degree even 25 to 80 times as long as in Case I.

In Case I the comparison between long- and short-term test relates to the end, in Case II to the start of the long-term test.

The writer concludes from the comparisons mentioned above:

1. The shrinkage of concrete significantly influences deformation under a long-term load. If this shrinkage occurs with a load increment of smaller intensity, as in Case II, the deformation will be less influenced by a long-term load of higher intensity than in Case I, where the shrinkage occurs with already comparatively high load intensities and the deformation dependent on load duration increases.

2. As is known the creep is dependent on load intensity as well as curing time of the concrete. So the creep will be the smaller, the more long-term load increments of lower intensity are substituted for those of higher intensity. Or, in Case I one may expect a higher creep percentage than in Case II. This works in the same way as the shrinkage just discussed.

3. It is known that strength and stiffness of concrete increase with the curing time. So a comparison between short- and long-term tests will be more conclusive when the short-time test is related to a longer curing period, as is done in Case I. On the other hand one has to consider that, generally, the short-term tests on which our code is based are made after a curing period that corresponds with the time of form removal on the job.

4. The discussor thinks that the time scheme for long-term tests should be adapted to building practice as much as possible, as with his tests described above.

5. About the influence of section shapes of the test objects (beams or slabs) on deformation under long-term load nothing is known up to now. The neutral axis of the slabs is not far from the underside of the plate, so that the total thickness of the plate shows nearly the same distribution of the concrete stresses as does the zone of pressure of the rectangular beam.

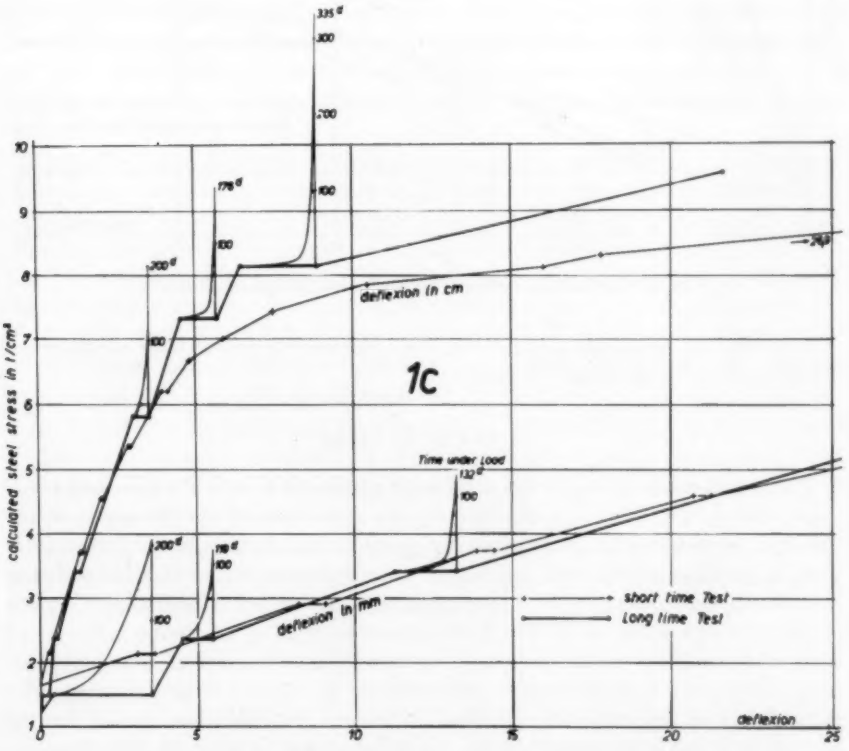


Fig. D—Deflection versus steel stress at midspan, Slab 1 c

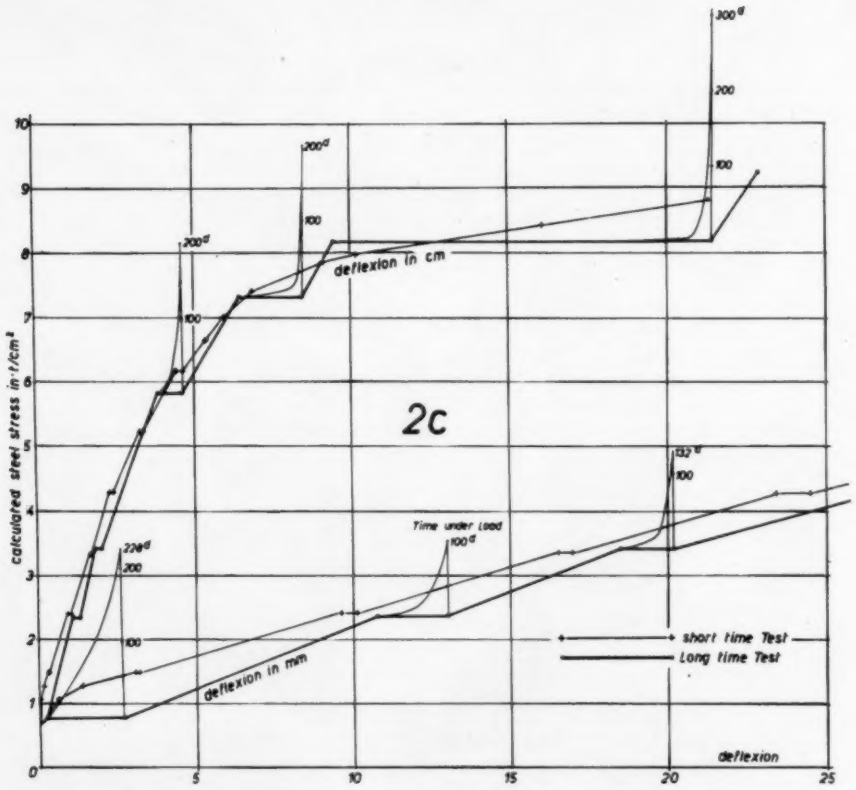


Fig. E—Deflection versus steel stress at midspan, Slab 2 c

In order to establish definitively the conclusions from these comparisons further extensive tests are necessary.

AUTHORS' CLOSURE

The relationship between the test results presented in this paper and those presented by Dr. Soretz is clarified by a comparison of the purposes of the two sets of tests. The authors' stated purpose was to determine influence of term of loading on ultimate strengths, ultimate compressive strains, ultimate curvatures, and moment-curvature relationships for beams somewhat typical of American practice, while Dr. Soretz' purpose was to determine influence of term of loading on crack formation for beams more typical of Austrian practice. Therefore, specimens and test methods of the two investigations differ radically, and information obtained in either investigation would be expected to have little relevance to the information desired of the other investigation.

Differences in test specimens are especially striking. While Soretz' beams contained only 5 to 14 percent, the authors' beams contained from 35 to 72 percent of the amount of steel required for balance (assuming that Whitney's value of $pf_y/f_c' = 0.456$ represents balance). Therefore, for Soretz' beams the concrete stresses are relatively low, resulting in low elastic and extremely low creep strains and a neutral axis which is relatively fixed and close to the extreme compressive fiber. The curvature of such a beam is primarily a function of the strain in the steel. Thus Soretz' curves essentially represent the stress-strain relationship for Torsteel 60 (elastic limit, 85,000 psi), while the authors' curves are a function of strains in both intermediate grade steel (elastic limit, 40,000 + psi) and concrete. Little correlation between these sets of curves could be expected, although each set is relevant to its respective test purpose.

As Dr. Soretz mentioned, a significant difference in method was in the timing of the short-term tests. By performing the short-term test at time of failure of the long-term-loaded beam the authors essentially eliminated variation in ultimate flexural behavior due to differences in the factors of concrete shrinkage, strength, or stiffness—factors whose effects on flexural behavior are largely known. Thus, only creep remained as a differentiating variable for each pair of beams. It is believed that had these three variables not been eliminated the test results would have lost much significance. The relative importance of these differentiating factors to the curves of the discussion is not known.

Regarding the difference in long-term loading increments, the authors believe that increments of from 10 to 30 percent of the ultimate load applied rapidly to a long-term-loaded beam would prevent any consistent determination of long-term ultimate strength or curvature, since such an increment would temporarily make the beam a short-term-loaded beam. That is, the values of ultimate moment and curvature would be significantly different for a beam which failed during the application of such an increment and a beam which failed during a constant-load period. As increments become smaller this difference would be reduced.

It should also be mentioned that curvature and strain are not proportional to deflection unless a structure is elastic. Therefore, post-elastic curvature and strain should be determined by direct measurement, preferably in a region of constant moment.

The authors appreciate the interest of Dr. Soretz in their work, as well as the information he has furnished on other aspects of a general area, which, all are agreed, needs much more investigation.

Discussion of a paper by John F. Brotchie:

General Method for Analysis of Flat Slabs and Plates*

By JAMES CHINN AND AUTHOR

By JAMES CHINN†

Mr. Brotchie has presented an interesting method of solution for flat slabs and plates which permits the use of various solutions to the problems of plates on elastic foundations.

Hertz's solution (author's Reference 1) for the point load has been used in the paper, and solutions for other load conditions are also applicable. Reference is made below to such solutions with which the writer is familiar.

In 1926, Westergaard[1] presented formulas and charts for deflections, moments, and stresses due to concentrated load in the interior, at the corner, and at an edge remote from a corner of a plate on elastic foundation. The solution for interior load was the Hertz solution and results were given in both equation and chart form. The solution for edge load was apparently rather complicated, so the presentation in equation form was limited, but the results presented in chart form were more complete. The results for the latter two loadings presented in Westergaard's paper can be applied to loads on the exterior edge and at corners of flat slabs. They cannot be used, however, for moments around columns which are monolithic with the slab.

In 1943, Westergaard[2] presented formulas for moments in plates on elastic foundations for various load distributions and various shapes of small, loaded areas. Mr. Brotchie suggests that the effects of load on various shaped areas can be determined by integration from the point load case. Although this is theoretically correct, it is difficult to integrate the mathematical functions involved over many of the common shapes of areas. The writer[3] found it necessary to expand the solution for the point load case into infinite series and integrate term by term for the case of a rectangular area. The first terms of the resulting series agree with Westergaard's solution obtained by an entirely different approach.

In 1951, Pickett and Ray[4] presented Newmark-type influence charts for determining numerical values of moments and deflections at an interior point, at an edge point remote from a corner, and at a point $0.5l$ ($0.5s$ by author's notation) from an edge of a plate on elastic foundation. Moments and deflections are for uniform load on any size or shape area which can be fitted on the charts to the proper scale.

*ACI JOURNAL, July 1957, *Proc.* V. 54, p. 31. Disc. 54-3 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

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Westergaard[1] also made recommendations for handling loads on very small circular areas. The solution for the point load gives infinite moments under the load. A true point load, however, is physically impossible; and even if it were possible, its effect at the middle surface of a plate, where the load is theoretically assumed to lie, would be that of a load distributed over some finite area.

Westergaard recommended that the load be assumed uniformly distributed over a circle of radius b where:

$$b = \sqrt{1.6a^2 + t^2} - 0.675t \text{ when } a < 1.724t \quad \text{(A)}$$
$$b = a \text{ when } a > 1.724t \quad \text{(B)}$$

a = actual radius of loaded circle
t = thickness of plate

The maximum moment is then given by:

$$M_{MAX} = 0.0458\,(1 + \mu)P \log_{10}\left[12(1 - \mu^2)\frac{s^4}{b^4}\right] \quad \text{(C)}$$

In 1946, E. Reissner[5] presented formulas for effects of loads on shallow spherical shells. The differential equation involved was shown by Bijlaard[6] to be identical with the one for plates on elastic foundations, so Reissner's results can be used in the latter case by merely replacing $(tE)/R^2$ with $k = D/l^4$, where R is the radius of sphere and other terms are as previously defined.

For the case of point load, Reissner obtained:

$$w = \frac{Ps^2}{2\pi D}\,\mathrm{kei}\,x \quad \text{(D)}$$

$$Q = -\frac{P}{2\pi s}\,\mathrm{ker}'\,x \quad \text{(E)}$$

$$M_r = -\frac{P}{2\pi}\left[\mathrm{ker}\,x - \frac{(1 - \mu)}{x}\,\mathrm{kei}'\,x\right] \quad \text{(F)}$$

$$M_t = -\frac{P}{2\pi}\left[\frac{(1 - \mu)}{x}\,\mathrm{kei}'\,x + \mu\,\mathrm{ker}\,x\right] \quad \text{(G)}$$

This solution is identical with the Hertz solution and is the basis for the author's Tables 1-4.

For the case of load uniformly distributed on a circle of radius $r_o = sx_o$, Reissner obtained:

$$w_i = \frac{Ps^2}{\pi D}\left[-\frac{\mathrm{ker}'\,x_o}{x_o}\,\mathrm{ber}\,x + \frac{\mathrm{kei}'\,x_o}{x_o}\,\mathrm{bei}\,x - \frac{1}{x_o^2}\right] \quad \text{(H)}$$

$$w_o = \frac{Ps^2}{\pi D}\left[-\frac{\text{ber}' x_o}{x_o}\ker x + \frac{\text{bei}' x_o}{x_o}\text{kei}\, x\right] \quad \text{(I)}$$

$$Q_i = -\frac{P}{\pi s}\left[\frac{\ker' x_o}{x_o}\text{bei}' x + \frac{\text{kei}' x_o}{x_o}\text{ber}' x\right] \quad \text{(J)}$$

$$Q_o = -\frac{P}{\pi s}\left[\frac{\text{ber}' x_o}{x_o}\text{kei}' x + \frac{\text{bei}' x_o}{x_o}\ker' x\right] \quad \text{(K)}$$

$$M_{ri} = -\frac{P}{\pi}\left\{\frac{\ker' x_o}{x_o}\left[\text{bei}\, x + (1-\mu)\frac{\text{ber}' x}{x}\right] + \frac{\text{kei}' x_o}{x_o}\left[\text{ber}\, x - (1-\mu)\frac{\text{bei}' x}{x}\right]\right\} \quad \text{(L)}$$

$$M_{ro} = -\frac{P}{\pi}\left\{\frac{\text{ber}' x_o}{x_o}\left[\text{kei}\, x + (1-\mu)\frac{\ker' x}{x}\right] + \frac{\text{bei}' x_o}{x_o}\left[\ker x - (1-\mu)\frac{\text{kei}' x}{x}\right]\right\} \quad \text{(M)}$$

$$M_{ti} = -\frac{P}{\pi}\left\{\frac{\ker' x_o}{x_o}\left[\mu\,\text{bei}\, x - (1-\mu)\frac{\text{ber}' x}{x}\right] + \frac{\text{kei}' x_o}{x_o}\left[\mu\,\text{ber}\, x + (1-\mu)\frac{\text{bei}' x}{x}\right]\right\} \quad \text{(N)}$$

$$M_{to} = -\frac{P}{\pi}\left\{\frac{\text{ber}' x_o}{x_o}\left[\mu\,\text{kei}\, x - (1-\mu)\frac{\ker' x}{x}\right] + \frac{\text{bei}' x_o}{x_o}\left[\mu\ker x + (1-\mu)\frac{\text{kei}' x}{x}\right]\right\} \quad \text{(O)}$$

The subscripts i and o refer, respectively, to inside and outside the loaded circle.

Eq. (C) can be derived from Eq. (L) or (N) for $x_o = 0$ and $x = 0$ if the series expansions for ber, bei, etc. are used. Tables for ber, bei, etc. are available in the author's Reference 5 and in Dwight's book.[7]

The tables in the author's Reference 2 can also be used if the following equalities are noted:

$$\text{ber}\, x = R_e\left[J_o\left(x\sqrt{i}\right)\right] \qquad \text{bei}\, x = -Im\left[J_o\left(x\sqrt{i}\right)\right]$$

$$\text{ber}' x = -R_e\left[\sqrt{i}\,J_1\left(x\sqrt{i}\right)\right] \qquad \text{bei}' x = Im\left[\sqrt{i}\,J_1\left(x\sqrt{i}\right)\right]$$

$$\ker x = -\frac{\pi}{2}Im\left[H_o^1\left(x\sqrt{i}\right)\right] \qquad \text{kei}\, x = -\frac{\pi}{2}R_e\left[H_o^1\left(x\sqrt{i}\right)\right]$$

$$\ker' x = \frac{\pi}{2}Im\left[\sqrt{i}\,H_1^1\left(x\sqrt{i}\right)\right] \qquad \text{kei}' x = \frac{\pi}{2}R_e\left[\sqrt{i}\,H_1^1\left(x\sqrt{i}\right)\right]$$

TABLE A—COMPARISON OF RADIAL AND TANGENTIAL MOMENTS DUE TO POINT LOAD AND TO LOAD UNIFORMLY DISTRIBUTED OVER A CIRCLE OF RADIUS, $r_o = 0.2\,s$

x	Radial moment		Tangential moment	
	Uniform load	Point load	Uniform load	Point load
0	0.1774	∞	0.1774	∞
0.1	0.1627	0.1529	0.1724	0.2323
0.2	0.1187	0.0984	0.1578	0.1774
0.4	0.0512	0.0460	0.1185	0.1232
0.6	0.0205	0.0180	0.0905	0.0924
0.8	0.0020	0.0007	0.0705	0.0714
1.0	−0.0095	−0.0105	0.0555	0.0561
1.5	−0.0225	−0.0229	0.0312	0.0314
2.0	−0.0239	−0.0241	0.0174	0.0175
2.5	−0.0205	−0.0206	0.0095	0.0095
3.0	−0.0155	−0.0156	0.0049	0.0049
3.5	−0.0107	−0.0107	0.0023	0.0023
4.0	−0.0067	−0.0067	0.0010	0.0010

A comparison between radial and tangential moments due to point load and to load on a circle of radius $0.2s$ is given in Table A and plotted in Fig. A. It is seen that distributing the load over a finite area reduces the maximum

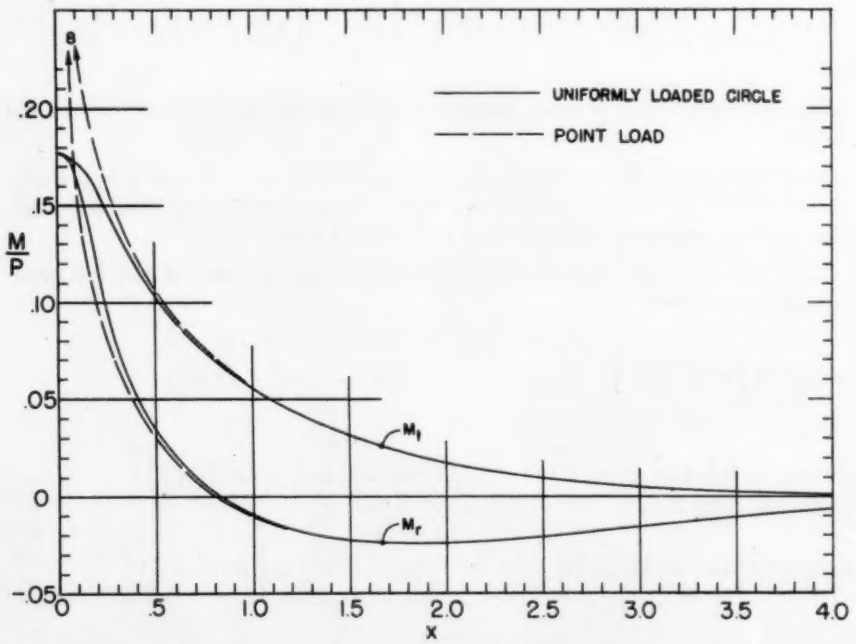

Fig. A—Comparison of moments due to point load and to load uniformly distributed over circle of radius $r_o = 0.2\,s$

moment to a finite value, but that moments a slight distance away from the load are nearly equal, as is to be expected from St. Venant's principle.

The writer does not recommend that reactions of columns and column caps integral with the slab be considered as uniform loads because of their restraint of rotation along their perimeters. Bijlaard[6] has shown, however, that the radial moment at the edge of a rigid insert is approximately equal but a bit higher than the moment at the center of the circle loaded uniformly with the same total load. The quantitative maximum moment from the latter loading, then, can be used if considered to act at the perimeter of column or cap.

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AUTHOR'S CLOSURE

The author wishes to thank Professor Chinn for his interesting contribution and particularly for his review of previous solutions for floating plates and of their relationship to solutions for flexure in spherical shells. An alternative method of presenting the author's theory would have been to consider the plate to have an initial hypothetical radius of curvature R, rather than to be floating on a liquid and in the limit where R approaches infinity, as when s approaches infinity, the analysis would be theoretically exact. However the theory is slightly simpler in its present form and the individual steps are much more easily visualized. The tables and formulae prepared for floating plates are applicable to the flexural analysis of spherical shells and the similarity now established between flexural stresses in flat suspended plates and in spherical shells under certain loading conditions may prove of value.

The paper under discussion is intended to be the first of a series on plate and slab analysis. Although originally broader in scope, only the basic theory and its application to uniformly loaded internal panels with circular columns. were detailed, in order to reduce the paper to a size suitable for publication.

Hence a number of the particular solutions referred to by Professor Chinn were reserved for inclusion in a later paper and so will be considered only briefly here.

The charts and formulae prepared by Westergaard and by Pickett and Ray, particularly those for an infinite slab, are applicable to the analysis together with an even more comprehensive set of influence charts for concrete pavements.* All these charts were developed for design of airfield and road pavements and are based, therefore, on a Poisson's ratio of 0.15. Two types of subgrade are treated, a liquid and an elastic solid; it is the charts for a liquid subgrade, where reaction at any point is proportional to deflection, that are directly applicable in this case.

Ordinary integration from the point loading case is simple only in certain cases as Professor Chinn suggests, but numerical integration, using the influence charts previously mentioned, may be readily used for distributed loads of any shape, and consists merely of drawing the load to scale on transparent paper, placing it over the chart, counting the number of squares covered, and substituting this number in the formula given. Both moments and deflections may be determined in this way.

The mathematical basis for the equations and tables is as follows: The deflection equation for Hertz's point loading case may be expressed in the form

$$w = \frac{s^2}{4D} \operatorname{Re}\left[P \cdot H_0^{(1)}\left(x\sqrt{i}\right)\right] \quad \text{.....(P)}$$

where Re is an abbreviation for "real part of" and $H_0^{(1)}\left(x\sqrt{i}\right)$ is a Hankel function, i.e. a zero order Bessel function of the third kind. $i = \sqrt{-1}$. Other Bessel functions may be used in the solution, but $H_0^{(1)}\left(x\sqrt{i}\right)$ is the most conveniently tabulated (author's original Reference 2).

Two additional cases, (1) deflection due to a radial moment (ring moment) applied around a circle of small radius, δx, and (2) deflection due to a symmetrical load (polar symmetry) on the floating slab, were derived by the author and may also be expressed in the form of Hankel functions.

Hence for ring moments

$$\lim_{\delta x \to 0}, \quad w = \frac{s^2}{4D} \operatorname{Re}\left[i\, C_1\, H_0^{(1)}\left(x\sqrt{i}\right)\right] \quad \text{.....(Q)}$$

where C_1 is a real constant which may be evaluated from the magnitude of the applied radial moment M_r. $(C_1 = -2\pi \cdot \delta x \cdot M_r)$.

*Pickett, G., Raville, M. E., Janes, W. C., and McCormick, F. J., "Deflections, Moments and Reactive Pressures for Concrete Pavements," *Bulletin* No. 65, Kansas State College, Oct. 1951, and associated set of 24 large-scale "Influence Charts for Concrete Pavements."

For the general case of any symmetrical load, of radius b, including the reactions of a concentrically loaded circular column, the deflection equation for the slab outside the load or column head is:

$$w = \frac{s^2}{4D} \cdot \mathrm{Re}\left[(C_2 + i\,C_3)\,H_0^{(1)}\left(x\sqrt{i}\right)\right] \quad \text{(R)}$$

where C_2 and C_3 are real constants. For small values of b, and this covers all practical column heads, C_2 may be assumed equal to P ($C_2 \rightarrow P$ as $b \rightarrow 0$) so that only one boundary condition at the column edge is required to determine C_3

By replacing the terms inside the square brackets by ζ in Eq. (P), (Q), and (R), general moment, shear, and deflection expressions are obtained.

ζ in each case has the useful property

$$\zeta'' + \frac{1}{x}\,\zeta' + i\,\zeta = 0 \quad \text{(S)}$$

Deflection:

$$w = \frac{s^2}{4D}\,\mathrm{Re}\,\zeta \quad \text{(by definition)} \quad \text{(T)}$$

Moments:

$$\text{tangential moment} = M_t = -D\left[\frac{1}{r}\frac{dw}{dr} + \mu\frac{d^2w}{dr^2}\right]$$

Substituting x and Re ζ

$$M_t = -\tfrac{1}{4}\,\mathrm{Re}\left[\frac{1}{x}\,\zeta' + \mu\,\zeta''\right]$$

Similarly

$$\text{radial moment} = M_r = -\tfrac{1}{4}\,\mathrm{Re}\left[\zeta'' + \mu\frac{1}{x}\,\zeta'\right]$$

M_t and M_r have been tabulated for $\mu = 0$; i.e.

$$M_t = -\tfrac{1}{4} \cdot \frac{1}{x} \cdot \mathrm{Re}\,\zeta' \quad \text{(U)}$$

and

$$M_r = -\tfrac{1}{4}\,\mathrm{Re}\,\zeta'' = \tfrac{1}{4}\,\mathrm{Re}\left[\frac{1}{x}\,\zeta' + i\,\zeta\right] \quad \text{(V)}$$

Shear:

$$Q_r = -D\frac{d}{dr}\left[\frac{d^2w}{dr^2} + \frac{1}{r}\frac{dw}{dr}\right] = \frac{1}{4s}\,\mathrm{Re}\left[i\,\zeta'\right] \quad \text{(W)}$$

Tables 1 to 4 were calculated for

$$\zeta = H_0^{(1)}\left(x\sqrt{i}\right) \quad \text{(X)}$$

and Table 5 for

$$\zeta = -4i\,H_0^{(1)}\left(x\sqrt{i}\right) \quad \text{(Y)}$$

Hence moments, deflections, and shears for the general case

$$\zeta = (C_2 + i\,C_3)\,H_0^{(1)}\left(x\sqrt{i}\right) \quad \text{(Z)}$$

are given by a linear combination of Eq. (X) and (Y).

For some small values of x in Tables 1 and 2, and to a lesser extent in Table 5, the same number of significant figures as given in the tables of $H_0^{(1)}\left(x\sqrt{i}\right)$, has been retained, so that the last figure was necessarily interpolated, and subject to the errors thus involved. Where greater accuracy is required the ker and kei functions in Reference 7 of the discussion could be used. The interval of tabulation is O.I x but intermediate values may be obtained from Taylor's theorem. ker x, ker′x, kei x and kei′x are tabulated, and higher derivatives may be obtained from Eq. (S); (e.g. let ζ = ker $x - i$ kei x, $\therefore$ ker″ $x = -1/x$ ker′ x − kei x, etc.).

Concentrated loads, including the effect of plate thickness are considered in a later paper. Note that Eq. (A) of the discussion was determined by Westergaard, for an unreinforced pavement slab where stresses on the face furthest from the load only are critical. It is not meant to apply to stresses on the loaded face, or where the load is close to a concentrated support.

The effect of plate thickness on the point load assumed to act at the center of a circular column need not be considered and it is assumed that Professor Chinn's discussion did not refer to this case. The point load and ring moment are only simplified components physically representing expressions containing the real and imaginary parts respectively of $H_0^{(1)}\left(x\sqrt{i}\right)$ in Eq. (R). They bear no direct relationship to the actual column reactions and the effect on them of plate thickness is irrelevant in this case. The effect of plate thickness on the actual column reactions is more intricate, but for all practical column heads the actual radius is sufficiently large to be assumed equal to the effective radius, i.e. $a = b$.

Professor Chinn's statement that "the radial moment at the edge of a rigid insert is approximately equal but a bit higher than the moment at the center of the circle uniformly loaded with the same load" is not sufficiently accurate in the present context, due in part to the different Poisson's ratio involved. The moment M_o at the center of the loaded circle (of radius b) = $P(m_t)_{x=b}$ (from Appendix B) for Poisson's ratio = 0, whereas for small values of b the radial moment (M_e) at the edge of the rigid insert is almost equal to $P\left[m_r + M_r\left(-m_t/M_t\right)\right]_{x=b}$ from the condition that the slope, and therefore the

tangential moment, is zero at this edge. m_t and m_r are given in Tables 1 and 2 respectively and M_t and M_r are given in Table 5.

For the example in the discussion where $b = 0.2$, $M_o = 0.177\ P$ whereas $M_e = 0.281\ P$ and as b approaches zero

$$\frac{M_o}{M_e} \rightarrow \frac{m_t}{m_t + m_r} \rightarrow \frac{m_t}{2m_t - \frac{1}{4\pi}} \rightarrow 1/2$$

The difference is of the same order when the effect of the other columns is included. M_o and M_e would be closer, however, for a material such as steel with a Poisson's ratio of 0.3, which was the case considered in Reference 6 of the discussion. Where the slab and column are integral there appears to be no good reason, in elastic analysis, for assuming a uniform reaction between them. The procedure for analysis is just as simple, and practically identical, (as shown in the author's example) for each boundary condition at the column head, and the most appropriate condition may be chosen in every case. As outlined in Appendix A, however, if the loading is increased until it approaches the ultimate flexural capacity of the panel, plastic yielding at the column face will allow the tangential moment at this face to increase until it equals the radial moment. Thus, it would appear that if plasticity is taken into account, any suitable boundary condition between $M_t = 0$ and $M_t = M_r$ might possibly be adopted for design, but this is outside the scope of the present discussion and will be considered in a later paper.

Discussion of a paper by Warrington G. Mitchell:

Effect of Waterproof Coating on Concrete Durability*

By CELSO A. CARBONELL and GEORGE L. OTTERSON

By CELSO A. CARBONELL†

Mr. Mitchell's article deserves a complete study. His recommendations could change the complete picture of concrete making.

Although surface protection of concrete does have its place in the concrete industry in general it can never replace the results obtained by improving concrete itself.

On p. 56 the author says, "It appears that durable concrete economically produced is more certain of realization through development of adequate surface protection than through slow and laborious improvement of the concrete itself." In my opinion such a statement should be based on very complete investigations, for it aims at the destruction of the principles of concrete making. After concrete has set there is nothing you can do to modify its characteristics. Certain limited factors such as wear resistance can be improved, but essentially concrete remains unchanged. In order to have concrete of optimum quality, and this includes durability, it is indispensable to proportion a concrete mix appropriate for the intended use of the concrete.

On p. 57 there is the following statement: "At present the engineer concerned with use of concrete as a construction medium has no alternative if local aggregates are inferior other than the costly importation of more durable materials; if he could choose on an economic basis between obtaining superior materials and providing an impermeable surface protection, design would be more rational, durable, and economical." In other words, Mr. Mitchell says that good concrete could be made without any consideration of its quality as long as its surface is later protected. Experience and research of many years in many countries and by thousands of engineers have proved the absolute opposite.

In considering just one of the many aspects of this problem, waterproofing, I will say that regardless of the type of surface protection, it will not give good results unless you have good concrete underneath. Water will go through defects in concrete. It must be a uniformly poor concrete and of an unusual porosity to let water through without the concrete having any "defects." This type of concrete is generally difficult to handle even while placing it. It

*ACI Journal, July 1958, *Proc.* V. 54, p. 51. Disc. 54-4 is a part of copyrighted Journal of the American Concrete Institute, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

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is essential to have good concrete in order to put a permanent coating over it. Mr. Mitchell's conclusions would perhaps be valid only if it were possible to encase all sides of a concrete structure, including the one exposed to earth in a permanently vaporproof and waterproof film, but then we would have to be sure that the concrete is bone dry before the film is applied; otherwise it would blister. Also the concrete never should crack. I sincerely hope to read more comments on this article dealing with such an important aspect of concrete making.

By GEORGE L. OTTERSON‡

The conclusions and recommendations contained in Mr. Mitchell's paper are indeed indicative of considerable constructive thinking relative to the goal of providing more durable concrete. There appear to be several potential applications of the use of waterproof coatings.

Synthetic coatings in tunnels, sluices, conduits, etc., subjected to high velocity flows, exceeding 50 fps, is one use which appears most merited. Such coatings are now being tested under prototype conditions. A similar application which is in extensive use involves the lining of concrete pipe for carrying of acid wastes.

The over-all and general application of coating concrete does not appear to be in the realm of extensive use due to lack of economic justification. Data as to costs per sq ft for a coating which will provide a definite period of complete sealing or protection are needed. Those data would permit an evaluation of the extent to which the use of a coating could be considered economical.

In summary, costs and effectiveness (duration of complete sealing or protection) are initially required as applies to prototype applications in conjunction with further considerations and evaluations of coatings which are currently available and/or are now being used. With such data, investigations and evaluations in a realistic manner should be executed most effectively by prototype and accelerated laboratory testing.

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Discussion of a paper by Paul F. Rice:

Design of Concrete Floors on Ground for Warehouse Loadings*

By H. S. HEAPS and AUTHOR

By H. S. HEAPS†

The results contained in Mr. Rice's paper are obtained on the assumption that the loaded concrete floor behaves as a thin beam. In view of the obvious application of the data to the design of floors it appears worthwhile to compare the results with those obtained from an independent treatment based on an exact analysis of the stresses within a thick slab.

In a previous paper‡ the general theory of elasticity was applied to determine equations for the stresses and displacements within a finitely thick elastic slab supported by a yielding base of known modulus k. The vertical stress transmitted through the lower surface of the slab was calculated for various values of the soil modulus. The equations may also be used to calculate the tensile stress at the upper surface of the slab.

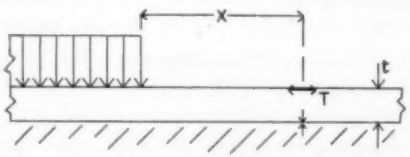

Fig. A—Loaded slab supported by base of modulus k

It follows from Eq. (5), (6a), and (6b) of the previous paper‡ that for a slab loaded as in Fig. A the tensile stress T at the upper surface of the slab is given by

$$T = \frac{2w}{\pi}\int_0^\infty \frac{2s(\sinh^2 2s + 4s^2) + c(\sinh 4s - 4s)}{F(s)} \frac{\sinh(2sx/t)}{2s}\, ds$$

where

$$F(s) = 2s(\sinh^2 2s - 4s^2) + c(\sinh 4s + 4s)$$

t = thickness of slab
x = distance from the edge of load to the point at which T is to be calculated
w = applied load per unit area
s = a variable of integration
$c = kt(1 - \sigma^2)/E$, where k is the soil modulus; σ and E are respectively Poisson's ratio and the elastic modulus of concrete

*ACI JOURNAL, Aug. 1957, *Proc.* V. 54, p. 105. Disc. 54-7 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Associate Professor, Nova Scotia Technical College, Halifax, N. S., Canada.

‡Heaps, H. S., "Transmission of Stress Through a Thick Slab Supported by a Yielding Foundation," *Canadian Journal of Technology* (Ottawa), V. 33, 1955, pp. 324-334.

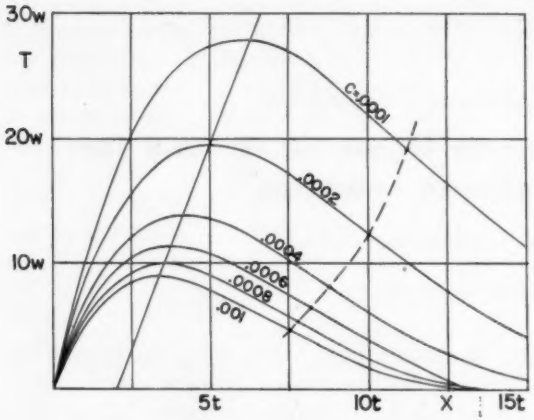

Fig. B—Dependence of the tension T at the upper surface of the slab upon the distance x and the parameter c

It may be observed that equations based upon the theory of thin beams contain $\lambda = (k/4EI)^{1/4}$ as a parameter while those based upon the general theory contain the parameter c. Neglect of Poisson's ratio is equivalent to putting $c = kt/E$. For a concrete slab 12 in. thick with $E = 3 \times 10^6$ psi and $k = 300$ lb per cu in., the value of kt/E is 0.0012.

Numerical integration of the above equation for T for various values of c and x leads to the graphs of Fig. B in which tensile stress at the upper surface of the slab is plotted as a function of distance from the edge of the load. The distance at which the maximum tensile stress occurs is shown by the continuous curve in Fig. C. The corresponding value of the maximum tensile stress is shown by the broken curve. It may be noted that in Fig. B the maximum points of the curves fit the empirical equation

$$T = 3.22\left(2\frac{x}{t} - 4\right)$$

For loads of the type considered in the author's Table 1, the critical aisle width to produce a maximum tensile stress is equal to twice the value of x shown in Fig. C. The agreement with the author's values is remarkable. For instance, if $t = 6$ in. and $k = 50$, then $c = 0.0001$ and Fig. C shows that $2x = 12.7t = 6$ ft 4 in. whereas the author obtained 6 ft 0 in. If $t = 12$ in. and $k = 50$, then $c = 0.0002$ and Fig. C shows that $2x = 10t = 10$ ft 0 in. as obtained by the author. If $t = 10$ in. and $k = 300$, then $c = 0.001$ and Fig. C shows that $2x = 6.8t = 5$ ft 8 in. whereas the author obtained 5 ft 7 in. The maximum tensions $2T$ are respectively $56.0w$, $38.8w$, and $17.8w$. These tensions become 225 psi when $w = 579$, 835, and 1820 psf. The author obtained 580, 825, and 1825 psf respectively.

The author stated that uplift of the slab off the ground cannot occur provided the slab weight exceeds 1/15 of the live load. Now uplift does not occur if the vertical stress transmitted through the lower surface to the soil

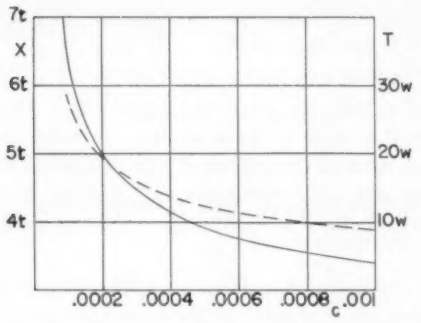

Fig. C—The continuous curve indicates the distance x at which the maximum tensile stress at the upper surface occurs. Broken curve indicates magnitude of this stress

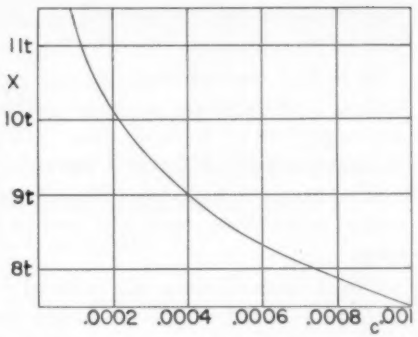

Fig. D—Curve indicates the distance x at which the vertical stress transmitted to the base becomes a tension

is compressive in nature. For the load shown in Fig. A the distance at which the transmitted stress ceases to be compressive was found in the previous paper* and is plotted as a function of c in Fig. D. It is supposed that the slab has negligible weight.

The points connected by a broken curve in Fig. B also indicate the conditions at which uplift first occurs. For example if $c = 0.0001$ then uplift first occurs when $x = 11.25t$; the tension at the upper surface of the slab directly above the point of uplift is then $19.0w$. Thus, if the slab had negligible weight then uplift would occur at the center of an aisle of width $22.5t$ and the tension at the point of uplift would be $38.0w$. It is apparent from Fig. B that, regardless of slab weight, there is never uplift before cracking in an aisle of critical width.

AUTHOR'S CLOSURE

The interest shown by Mr. Heaps is gratifying. Confirmation of the original results by a third independent method of analysis is appreciated. Values in the original table were computed by slide rule and rounded off to the nearest 5 psf. The close agreement with moment values, critical aisle widths, etc., noted in the discussion as remarkable was to be expected. The analysis employed by Mr. Heaps is a more general solution, but use of a Poisson's ratio of zero makes it equivalent to the much simpler analysis summarized in the paper.

The apparent error noted in the discussion relative to uplift is entirely apparent, not real. The statements in the discussion are all correct, but the statements in the original paper are also all correct. In preparing the paper no effort was made to discuss or illustrate mathematical results irrelevant to practical design. The same locations of uplift shown in Fig. B and D of

*See third footnote p. 799.

the discussion can be secured by plotting the trigonometric-exponential portion of the equation for deflection (p. 109 original paper).

It is true that no full symmetrical loading can cause uplift in an aisle of critical width before cracking. It is also true as originally stated that the slab weight must be more than 1/15 the live load to avoid uplift. It must be recognized that an actual warehouse operation will not conform to idealized assumptions of loading. Storage areas may be intermittently loaded, unloaded cross aisles exist, and load in loaded areas is seldom symmetrical very long.

The 1/15 coefficient was derived for the point of maximum uplift as given by superposition of opposing fully loaded areas a critical distance (for maximum uplift) apart. The resulting equation is $ky = 0.0671\ w_{LL}$, and therefore $w_{LL} = 14.9\ w_{DL}$ will begin to cause uplift. Thus, the fact that visible uplift was observed in, and was the most obvious symptom of, a floor failure is theoretically verified.

The principal consideration in choosing between the simple analysis developed in the paper and the general solution developed by Mr. Heaps which can include the effect of Poisson's ratio should be the accuracy of results related to accuracy of assumptions. For ordinary structural analysis, assumptions are quite crude and there exists much justification in neglecting Poisson's ratio.

L'Hermite states that Poisson's ratio varies with proportions, age, and moisture of concrete.[1] Couard states that Poisson's ratio is a variable with amount, type, and direction of stress,[2] and of course we have both tensile and compressive stress varying with depth and from two directions. Westergaard and Slater show small change in theoretical analyses for Poisson's ratio between zero and an extreme of 0.3 for supported slabs.[3]

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Discussion of a paper by Gene M. Nordby and William J. Venuti:

Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete*

By A. M. OZELL and AUTHORS

By A. M. OZELL†

The authors are to be congratulated on the great care taken in planning and conducting the tests. Some of the conclusions are a valuable contribution to the understanding of the behavior of prestressed concrete members subjected to repeated loads.

The size of the specimens used in these tests was extremely small and caution should be exercised in applying the results of such tests to full-size members.

No detailed description of the method of attaching the SR-4 strain gages to the strands is given. It is important to note that the strands are made of wires which have internal stresses caused by the manufacturing process, and the elastic properties are not uniform. Extremely small gages would have to be used to eliminate the effect of the helical wire surface, because otherwise the readings would include strains in three dimensions. On the other hand, strains measured by small gages would be affected by the localized inelastic areas; hence, numerous small gages should be used to arrive at average strain values.

No mention is made of permanent set which would be exhibited by beams subjected to repeated overloads. Tests conducted at the University of Florida‡ have repeatedly shown the importance of permanent set which is usually a warning of possible failure. Incidentally, Florida tests have indicated no measurable increase in the stiffness of prestressed concrete members.

The increase in the stiffness of such members would only result in beams in which stress level caused by prestress is very low and, at such stresses, creep in the concrete would be negligible whereas the increase in E due to aging might be as much as 30 percent.

In the tests reported for Beams 1A, 1B, 3A, and 3B, the maximum concrete stress at the support was only 0.28 f'_{ci} compared to 0.50 to 0.60 f'_{ci}, which are usually used as the desirable and efficient prestressing level. It is

*ACI JOURNAL, Aug. 1957, *Proc.* V. 54, p. 141. Disc. 54-10 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

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‡Ozell, A. M., and Ardaman, E., "Fatigue Tests of Pre-Tensioned Prestressed Beams," ACI JOURNAL, Oct. 1956, *Proc.* V. 53, pp. 413-424.

believed that the increase in stiffness should be attributed to the increase in E, due to aging, rather than be considered as a natural characteristic of prestressed concrete beams subjected to repeated loads. At stress levels of 0.55 f'_{ci}, creep would offset the increase in E and no increase in stiffness would result.

Despite all the care exercised during the tests, it seems improbable that measurements of slip amounting to 0.0002 in. could be reliable since 0.0001-in. dial gages would be extremely sensitive to external movements and, therefore, one division reading should not be used as a standard.

Some field studies* conducted in casting yards to determine the length of embedment for bond for 7/16-in. strands for full-size bridge girders indicated that full bond developed within 12 to 20 in. from the end of the girder. Laboratory tests with more accurate instrumentation may change these figures. Conclusions drawn from tests of very small beams, however, should be treated with caution.

AUTHORS' CLOSURE

The authors welcome the queries of Professor Ozell since they give an opportunity to elaborate more fully on the research techniques and conclusions drawn.

The authors concur that specimens were small and that the results should be used with caution. However, such scaling down in experimental investigations is not unusual, and much of our reinforced concrete theory is based on such tests. The size of the beams did approximate the size of the stems of some double-T sections in production at the time of the tests, however. The authors decided in this initial study to test a large number of small beams rather than a few large beams as an economical exploratory investigation.

To collect data as accurately as possible it is necessary to perform tests under ideal conditions. Laboratories afford this condition but limit the size of specimens that can be accommodated because of testing machine capacities, handling equipment, etc. Tests in the field made on full-size specimens yield more realistic data than scaled beams but recording equipment and loading methods are rather crude. The tests now being carried out at Lehigh University may clarify this problem.

The strain measurement problem is, indeed, a difficult one which will probably not be completely solved in the near future. The gages used were SR-4 (A-12) which have a 1-in. gage length and 3/32-in. trim width. With proper softening by cement, it was possible to shape the gage to the helical path of the strand. It was necessary, of course, to develop a great deal of skill in attaching the gages and thus very few gages failed to function during the test. The gages were first cushioned with a pad of friction tape and then waterproofed with "petrolastic" (a product of Standard Oil of California).

*Ozell, A. M., and Givens, W. D., "Curing Methods and Duration Studies of Pretensioned Units," *Bulletin* No. 82, Florida Engineering and Industrial Experiment Station, Aug. 1956.

All wiring was also waterproofed from the gage to the exterior of the beam. Incidentally, the method used was similar to that used with great success by Janney* in his well-known investigation on bond which was carried on at about the same time as this investigation. He used "Product 2300" (Shell Oil) for waterproofing.

The authors believe that the gages recorded the strain closely for several reasons. First, the gages were attached while the strands were under high tension and the individual wires were well seated. Secondly, any small torsional stresses and bending stresses due to movement of an individual wire are minimized by the small moment of inertia of the wire so that the strain measured is very nearly the principal strain.

Thirdly, the seven-wire pretensioning, prestressing strand is stress relieved, thus no residual stress due to the manufacturing process should exist. Finally, the angle of lay of the wire in the strand was 13 deg which at the most could result in an error of 2.5 percent in stress computation. This difference was considered when computing Δf_s by using a modified E_s for certain calculations. Unfortunately, the internal side of the wire could not be instrumented nor could each wire of the strand. Nevertheless, the gages gave accurate information on stress increase above initial prestress level which was their principal use.

No accurate measurement of set was taken during the tests. During the fatigue loading some set was noted (less than 0.1 in. at center) but did not appear to affect their performance. A significant difference in Professor Ozell's fatigue machine (constant deflection) and the authors' (constant load) made it unnecessary to use permanent set measurements to detect failure. All fatigue failures occurred within 1000 cycles of load (usually less) after failure commenced, since the load followed the permanent set of the beam and resulted in rapid failure. This resulted in a more precise counting of the number of fatigue failure cycles than was obtained by Professor Ozell in tests described in his recent paper† where extrapolation of curves was necessary to determine the number of failure fatigue cycles. Unfortunately, both investigations were simultaneous so that the authors could not benefit from Professor Ozell's results in carrying out their tests.

The authors agree with Professor Ozell's comments on the accuracy of the 0.0001-in. dial gages. Extreme care was necessary to record slip. No slip of strand was reported, however, unless a complete slip history was recorded. Despite the extreme care taken (polished strand ends) the "wobble" in the needle was ± 0.0001. Slip was thus only recorded when 0.0002 in. movement was recorded followed by 0.0005 to 0.0010 in. more on the next load increment. In every case the slip eventually outran the capacity of the gage. The authors are aware of the shortcomings of these gages and interpreted the gages accordingly. The question about the gages reminds

*Reference 2, original paper.
†Ozell and Ardaman, *op. cit.*

the authors of the old "saw," "Everyone believes a theoretical solution except he who makes it, while everyone doubts experimental data except he who takes it." As every experimenter knows, there are times he must rely to a certain extent on his extra-perceptional sense and in this case the *internal noise* together with dial readings at the time indicated slip had occurred.

The authors do not agree with Professor Ozell's hypothesis of the stiffening of the beams. The beams in which this effect was most pronounced were well over 1½ years old, and each beam of a matched pair was tested within 2 weeks (usually less) of its mate. This eliminates any great change in E due to aging. In addition, numerous cylinders were used to determine the concrete properties at the various time intervals. Very little change in E took place after the first 90 days since the concrete was dry (and kept dry by low Colorado humidity). The authors must apologize that these details could not be outlined in the original paper due to limited space. Certainly, at the stress levels used, the steel could not contribute to the added stiffness. Therefore, the concrete under extensive repeated load must be the cause.

The authors call attention to other evidence that such stiffening is possible. Rowe[13] in a recent appraisal of research on the fatigue of prestressed concrete said: (article did not include Professor Ozell's or the authors' work) "In some instances a noticeable work-hardening effect in the concrete was observed during fatigue loading in the elastic range. This resulted in increased stiffness, and suggests that the normal elastic fatigue loading could be sustained indefinitely by the concrete." Guyon* too, describing such action in plain concrete, says " . . . concrete is thus work-hardened under repeated loading and the strain diagram between the upper and lower limits tends to a certain definite form which does not change with further loadings." In fact, this process is well established, having been first noted by Van Ornum.[14,15] The process was described in more detail particularly by Probst and his associates[16,17,18] and by Graf and Brenner.[19,20] Unfortunately, the truly fundamental research on plain concrete is not extensive for high strength concrete on this point, nor is it clear how the work-hardening affects beam action. Such action may depend upon type of aggregate and mix proportions, as well as loading conditions. However, it is clear that the shape of the stress-strain diagram for concrete may vary under repeated load.

The authors must also disagree with the implications in the reported full bond development within 12 to 20 in. from the end of a beam using 7/16-in. strands. There is a general misunderstanding of what is meant by the anchorage length. That reported by Professor Ozell is the initial anchorage length L_e developed in retaining the initial prestress in the prestressing strands. Another anchorage length L_u is necessary, however, in the ultimate design of prestressed members, since these members may fail in flexure, shear, or bond, and these factors should be coordinated in such a design by understanding

*Reference 1, original paper, p. 57.

the mechanism of failure of each. Certainly, this feature will provide further basis for research, since the papers of Thorsen, Echols,* and the authors have served only to point out the problem.

*See References 3 and 4, original paper.

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Discussion of a paper by Frank Kerekes:

Review of Changes in the ACI Building Code Requirements for Reinforced Concrete*

By JAIME de las CASAS†

This paper by Dean Kerekes is the most precise comment the writer has seen on the ACI Building Code. With his great experience as chairman of the committee, he presents a positive philosophy for understanding the Code, and the writer is in complete agreement with this part of the paper, but concerning the section "Summary of Code Revisions," the writer thinks it would be opportune to consider some of the statements made by others.

Inconsistencies in "design of flat slabs" noted by Alfred Zweig[1] indicate the necessity of a revision.

Regarding "reinforced concrete columns and walls," the present Committee 318 remarks[2] that "At that time [in the 1936 revision of the Code], the Code left bending in the working stress category, so that a member subjected to combined bending and direct stress was a hybrid, its bending resistance being approximated by the working stress method and its compression by the ultimate strength method." Unfortunately in ACI 318-56 this inconsistency persists as it is shown[3] by ACI-ASCE Committee 327: "In his discussion, Professor Siess ably brings out the variable factor of safety inherent in the present method of designing columns. If his discussion had been pursued further to the case of eccentrically loaded columns it would be seen that this variation in the factor of safety is even more marked for the case of combined bending and axial load, *ranging from an extravagantly high value of 4 to in some cases less than 2.*" (Writer's italics.)

Regarding "shear and diagonal tension, bond and anchorage," Morrow and Viest[4] say, "Considering the brittle nature of diagonal tension failures, it may be concluded that the ACI 318-56 provisions for shear in beams without web reinforcement did not provide an entirely adequate factor of safety for several of the beams listed in the Table 11; nevertheless, these provisions represent a definite improvement as compared with ACI 318-51."

Taking into account the extensive task accomplished by the former Committee 318, it would be unfair to criticize the good work, but it is also significant to consider the aforementioned inconsistencies in contemplated future Code revisions.

*ACI JOURNAL, Sept. 1957, *Proc.* V. 54, p. 185. Disc. 54-11 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

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Apart from the good trends stated by Dean Kerekes, the writer believes it would be convenient to recognize the great importance of *ultimate strength design*, adopted as the only design method in several foreign countries, and to consider the possibility of orienting the future Code in that direction.

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Discussion of a paper by F. T. Mavis and M. J. Greaves:

Destructive Impulse Loading of Reinforced Concrete Beams*

By LOUIS BALOG, E. H. FEY, K. HAJNAL-KÓNYI, ROBERT J. HANSEN, BRUCE G. JOHNSTON, NATHAN M. NEWMARK, and MERIT P. WHITE, A. HELFGOT, K. E. McKEE, F. WALLEY, and AUTHORS

By LOUIS BALOG†

The U. S. Atomic Energy Commission stated in 1950 that tests have shown that structural- or intermediate-grade steel is better than hard-grade steel for reinforcement against impulse loads. The authors begin and end with the statement that in an exhaustive search of published literature they did not find experimental data on tests of reinforced concrete beams that would support such an opinion.

On Feb. 1, 1937, the writer presented a report on the computation of the effect of impact on structures to the New York District of the Corps of Engineers. In this report the authors' Reference 4 (1936) and Reference 83 (1936), were quoted. Reference 4 stated on p. 341:

> *The tests showed that the yield stress of St 37 (a structural-grade steel similar to ASTM A 7) is substantially higher at impulse load than at static load. At a steel stress of 49,780 psi due to impulse load the beam still behaved as completely elastic, although the average yield stress of this steel, as established by tension test, was 34,140 psi. High-grade steels (Isteg, St 55, St 80) showed no increase in yield stress or only to a lesser extent.*

Reference 83 summarized the test results in Table 25, p. 79; these are repeated in Table A, which contains average values and shows the superiority of the structural-grade steel St 37 over the high-strength steels in carrying impulse loads.

The test beams were about 8 x 10 in. in section and had 1.2 percent reinforcement of steel St 37 and the equivalent amount from the other steels. The beams were $1\frac{1}{2}$ years old when tested; the prism and cube strengths of the concrete were 2990 psi and 5120 psi, respectively.

The authors indicate the load, shear, and moment distribution for the dynamic test and longitudinal steel requirement at the temporarily tensioned side of the beam, but not the amount of the web steel in the specimens and what the "whiplash" steel should be. A great deal of effort and public funds were spent for the solution of efficient dynamic design during recent years.

*ACI JOURNAL, Sept. 1957, *Proc.* V. 54, p. 233. Disc. 54-14 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 9, Mar. 1958, *Proceedings* V. 54.

†Consulting Engineer, Binghamton, N. Y.

TABLE A—STATIC AND IMPULSE LOAD TESTS OF BEAMS REINFORCED WITH VARIOUS GRADES OF STEEL

1	2	3	4	5	6	7
Steel	Tensile yield stress, psi	Static bending stress, psi	(2)/(3)	Impulse bending stress, psi	(5)/(2)	(6)/(4)
St 37	34,140	38,820	1.14	49,070	1.44	1.26
Isteg	57,600	68,980	1.20	73,250	1.27	1.06
St 55	55,900	64,000	1.14	74,670	1.33	1.17
St 80	68,340	83,920	1.23	94,595	1.38	1.12

The authors concluded that hard-grade steel assured the largest static and dynamic, elastic and ultimate, load capacity.

The history of reinforced concrete structural failures indicates, however, that proper attention to the details of design is the most efficient safety measure. The yield point of longitudinal steel in the Air Force warehouse girders which collapsed under dead load exceeded 100,000 psi; the concrete, averaging 4600 psi, was of high and uniform quality. Actually, if this steel in the girders had been distributed properly, they might have possessed the strength indicated by the authors. Research exists only for the improvement of design; without the complete description of the reinforcement the grade of steel used is meaningless.

By E. H. FEY*

This paper deals with the comparison of hard-grade and intermediate-grade steels for reinforcement of concrete beams subject to dynamic loads.

A similar comparison has been conducted in the shock-loading machine at Massachusetts Institute of Technology, with results presented in Chapter III of a report by Galletly, Matsuda, and Ofjord.† The conclusions of this MIT report are contrary to the conclusion of the authors—namely, that hard-grade steel is superior to intermediate-grade steel for reinforcement in beams subject to blast forces and destructive impulsive loads. Curiously, this MIT report is not listed in the authors' extensive bibliography. Possibly this is the report alluded to in the article as supporting the opinion which the authors have attempted to disprove.

The authors have succeeded in developing a dynamic loading device in which hard-grade beams outperform similar intermediate-grade beams. The loading device utilizes a pair-wise testing setup which applies equal impulses to two beams. This appears to be a fair test since, if one beam fails and the other does not, it means that the beam which did fail could not withstand the impulse which the other beam resisted.

However, the test results presented appear to indicate that there were some peculiarities of the loading which prohibit broad generalization of the

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†Galletly, G. D., Matsuda, F., and Ofjord, A., "Behavior of Structural Elements Under Impulsive Loads II," Massachusetts Institute of Technology, Cambridge, Mass., Nov. 1950 (a report submitted to New England Division, Corps of Engineers, Department of the Army, under Contract No. DA-19-016-eng-239).

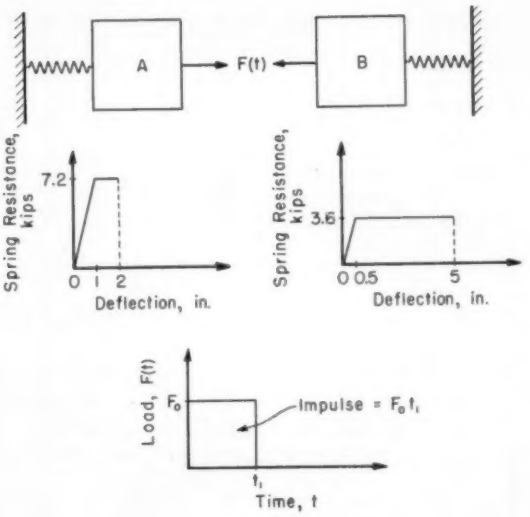

Fig. A—Theoretical comparison of impulse loading of similar structures

test results. For the elastic range, the natural period of vibration of the beams in the first mode is estimated (using the static load-deflection data in Fig. 8 and 12) to be 0.026 sec.

From Fig. 13, it can be seen that a significant load was applied for a time equal to twice this value and perhaps even longer. The possibility exists that, had the load lasted only a small fraction of the natural period, the conclusions of the test would have been reversed, i.e., the hard-grade beams might have failed while the intermediate-grade beams did not. Shorter duration loads could be obtained in the existing load fixture by increasing the stiffness of the spring.

To show that such a possibility exists, consider a simple case of two structures: A and B, shown in Fig. A. Equal impulses in the form of a step function lasting for a time t_1 are applied to each structure. The two structures have the same stiffness in the elastic range and the same mass. Hence, they will have the same period of vibration in the elastic range, say 0.025 sec. Structure A has a yield resistance of 7200 lb, a yield displacement of 1 in., and fails at a displacement of 2 in. Structure B, on the other hand, has a yield resistance of only 3600 lb, a yield displacement of $\frac{1}{2}$ in., and fails at 5 in.

For a given duration of the load, the value of the impulse can be adjusted so that one or the other (or both) of the beams fail. The beam that can withstand the greater impulse at that duration will be the one that doesn't fail and will be judged the stronger of the two for that load duration.

The values of the impulse required to produce failure in each of the structures at various durations, t_1, of the step-function load are plotted in Fig. B.

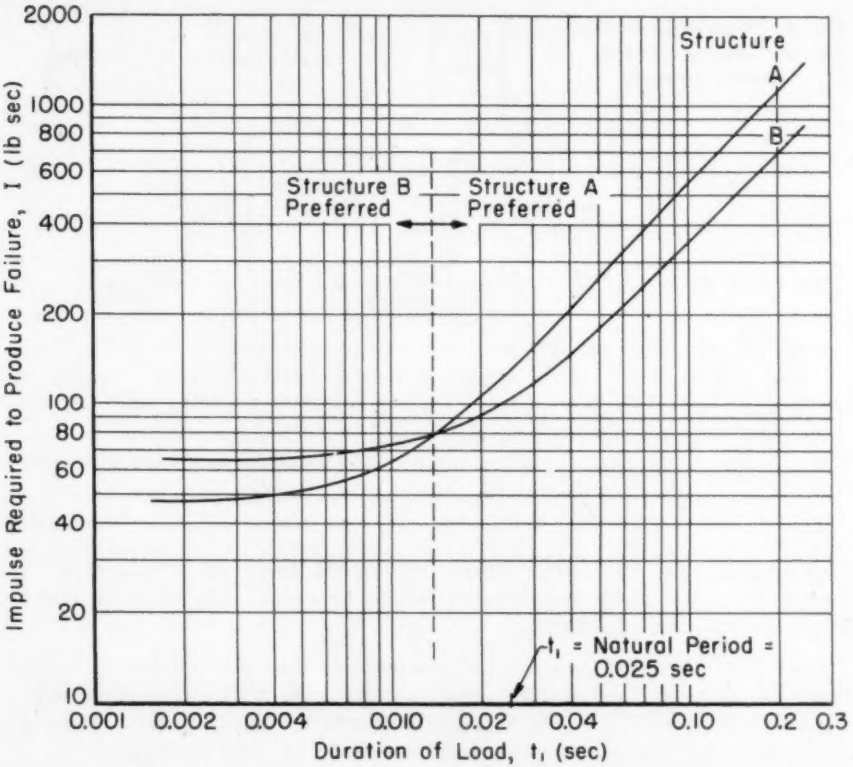

Fig. B—Values of impulse required to produce failure in Structures A and B

Note that, for durations greater than about 0.6 of the natural period ($t_1 >$ 0.014 sec), Structure *A* outperforms Structure *B*. However, for tests conducted at durations less than approximately 0.6 of the natural period, the reverse is true, i.e., Structure *B* is considered preferable.

The region of durations where Structure *A* is preferable may be termed the long-duration range. In this range, the value of the yield resistance governs the behavior of the structures. The extreme case in this range is a static load suddenly applied. For such a case, one would expect the structure with the greatest yield resistance to be superior.

Conversely, in the region of short-duration loads, the energy absorption characteristics of the structures govern their behavior. The extreme case in this region is a pure impulse. In such a case, the structure with the greatest area under its associated resistance-deflection curve is superior regardless of the value of its yield resistance.

All of the authors' tests appear to be in the long-duration range. Consequently, short-duration load tests would also be required to validate the

authors' contention that hard-grade steel is preferable to intermediate-grade steel as reinforcing for beams subject to blast loads—unless their conclusions are accordingly modified.

By K. HAJNAL-KÓNYI*

The authors are to be congratulated on the ingenious method applied to their tests and their conclusions are convincing as far as they go. However, in view of the great importance of the subject and our limited knowledge of it, in spite of the vast amount of literature included in the bibliography, it would be desirable if they extended their research.

In the present research the same percentage of reinforcement was used in every pair of beams for both grades of steel. In the closure to a previous paper on the same subject† it was suggested that the amount of low-yield-strength and high-yield-strength steel used in a slab or beam should be about inversely proportional to the yield strength. Consequently, it would be of great interest if further tests were made in which the percentage of hard-grade steel would be about half of that of intermediate-grade steel in the companion beam. While the present tests have proved the superiority of beams with hard-grade steel compared with beams having the same percentage of intermediate-grade steel, it has yet to be investigated whether a beam with reduced percentage of hard-grade steel is equivalent to a comparable beam with intermediate-grade steel.

The tests were confined to one quality of concrete and one percentage of reinforcement. The tensile reinforcement index $q = pf_y/f_c'$ was $(0.0162 \times 40{,}000)/6500 = 0.100$ and $(0.0162 \times 75{,}000)/6500 = 0.187$ for the two grades of steel respectively. In the previous research referred to, the extreme values of q were 0.093 and 0.172 for intermediate-grade steel and 0.178 and 0.328 for hard-grade steel. It would be desirable to cover a wider range of the tensile reinforcement index both upward and downward.

By ROBERT J. HANSEN,‡ BRUCE G. JOHNSTON,§ NATHAN M. NEWMARK,** and MERIT P. WHITE††

In reviewing the paper by Mavis and Greaves we find that for the reported static load-deflection curves shown in Fig. 8 and 12, for beams reinforced with intermediate-grade and hard-grade steel, respectively, the dynamic strengths theoretically should be roughly in proportion to the static strength for the load-time curves used by the authors. It is not surprising therefore

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†Mavis, F. T., and Richards, F. A., "Impulse Testing of Concrete Beams," ACI JOURNAL, Sept. 1955, *Proc.* V. 52, pp. 93-102. Discussion pp. 1153-1155.
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that the authors' tests substantiate the above-stated conclusion. What is surprising is the authors' observation that for equal bar areas only "about 13 percent more initial load was required to start the damage in a hard-grade beam than in an intermediate-grade beam." On the basis of this observation we can only conclude that hard-grade reinforcement certainly cannot be substituted for intermediate-grade reinforcement for dynamic loads on the basis of equal static strengths.

The authors make the statement, which they claim they disprove, that "structural-grade or intermediate-grade steel is also better than hard-grade steel as reinforcement to resist heavy blast forces and destructive impulse loads." This statement was first developed as a design criterion during the early part of World War II in regard to loadings from high explosive bombs and applies strictly to "impulsive" loadings where the total duration of the loading is less than approximately 0.3 times the natural period of vibration (in this case, of a cracked concrete beam). In no case was the statement intended to be applied to beams of other than the same static yield strength.

If the total duration of the loading in the authors' case had been 0.006 sec or less, one would have expected the energy absorbing capacities of the beams to govern their dynamic strengths. However, for loadings of the type and duration used by the authors, theoretical calculations indicate that the dynamic strengths of the beams with intermediate-grade steel should be nearly equal to the static yield strengths increased by the speed of loading effect, amounting to about 30 to 50 percent over the static yield strength. For the beams with hard-grade steel the expected bonus is smaller.

Engineers must be careful about drawing sweeping conclusions from inadequate tests. It has not been proved that in all cases hard-grade steel can be substituted for intermediate-grade. Obviously, tendencies toward brittle fracture were not even considered by the authors. Nor were the different mechanisms of failure considered for beams designed to have different strengths. It is believed that engineers responsible for dynamic design will want a more adequate basis for departing from design criteria which provide assurance of ductile behavior in structures required to resist blast loading.

By A. HELFGOT*

The paper contains many important facts as to the dynamic behavior of reinforced concrete beams. However these beams were identical in all but the grade of the reinforcing steel. The authors found that "hard-grade beams outperformed the intermediate-grade beams consistently." This important fact, illustrated by significant curves, indicates that to obtain an equivalent performance it is possible to diminish the percentage of hard-grade steel. And this is precisely the point I should like to make clear—what percentage reduction of hard-grade steel can be made to get strengths equivalent to

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that of the original amount of intermediate grade steel? If the authors have these data, it would be quite interesting to have them published in order to draw all the technical as well as economical consequences.

By K. E. McKEE*

This paper was concerned with the relative merits of hard-grade steel compared with structural- or intermediate-grade steel for use in reinforced concrete beams subjected to dynamic loads. Since the design criteria for resisting dynamic loads are unquestionably different from those for static loads, there is a definite need for this information.

The authors claim that hard-grade steel is superior to the structural- or intermediate-grade steel for dynamic as well as static loading. This may be correct, but the results presented do not seem sufficient to support this contention. Since a complete set of experimental data which would allow a conclusion to be made is not available, the comments in the following paragraphs are directed only at specific aspects of the paper where further clarification would appear to be required.

The equation of motion for the equivalent single degree of freedom system representing a reinforced concrete beam may be written as:

$$M_e \ddot{x} + R_e(x) = F_e(t) \quad \text{.....(A)}$$

where

M_e = equivalent mass

$R_e(x)$ = equivalent dynamic resistance as a function of displacement, x

$F_e(t)$ = equivalent force as a function of time, t

x = displacement of characteristic point

$\ddot{x} = \dfrac{d^2x}{dt^2}$ = acceleration of characteristic point

t = time

The assumptions implicit in Eq. (A) include: (1) The resistance function is independent of time. (2) The forcing function is independent of displacement. (3) No damping is present in the system, i.e., terms involving $\dot{x} = dx/dt$ do not appear.

For static loading $R_e(x)$ would be, by definition, the actual static load displacement curve, while for any other loading $R_e(x)$ would be expected to differ depending on the load-time history. The actual dynamic resistance function, $R_e(x)$, must be computed from Eq. (A) using measured forcing functions (load-time curves), accelerations, displacements, and times. The basis for comparison of the ability to resist failure due to impulsive loading should be these resistance functions. For the most severe loading condition, an initial impulse, the area under the $R_e(x)$ curve to the displacement where $R_e(x)$ returns to zero is proportional to the maximum impulse which can be resisted without failure.

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The term "initial impulse" is used here to refer to the area under the force-time curve when the duration is very short compared to the period of the beam. Under these circumstances the significant feature of the loading is this initial impulse, while the detailed features of the force-time curve have little, if any, influence on the response. Not only does initial impulse represent the most severe condition of dynamic loading, but it has the additional advantage of being one of the simplest to handle analytically. For other types of dynamic loading care must be taken in evaluating test results, since more extreme loading conditions, e.g., the initial impulse, are possible.

The experimental data as presented in the paper are not sufficient for computation of the $R_e(x)$ as defined in this discussion. It is difficult to understand how rational conclusions can be based on Fig. 8 and 12. There would appear to be no basis for comparing stresses computed from these curves or for rating the effectiveness of various types of steel. The inertial effects must be taken into account since the load-deflection curve in itself has no meaning. Any arbitrary load-deflection curve can be imposed on a system. As a demonstration of this, consider a mass sliding on a frictionless surface and having an applied horizontal force. Any force-displacement function could be applied to this system, assuming there were no other limitations imposed. This is admittedly a simple example, but it appears that the authors are using equally arbitrary force-displacement curves on Fig. 8 and 12.

Several observations are made by the authors regarding the curves of Fig. 7. It is not clear what is being considered. Up until 0.010 sec (½ in.) the two deflection-time curves are approximately equal—this means only that up to that deflection the resistance curves, $R_e(x)$, are very nearly the same. "Equal damage" would appear to have no meaning in this situation. Beyond this point it should be noted that the responses of the two beams differ substantially. Here again the inertial effects do not appear to have been taken into consideration before making comparisons.

Pairwise dynamic testing, as applied by the authors, cannot be used as a basis for comparison for a general dynamic load. The data collected in these tests if used in connection with Eq. (A), or a similar equation, would be extremely useful. However, there is no basis for considering this type of pairwise testing to be valid for direct comparisons of relative beam strengths except for the specific force-time function which is applied. For static loading this is obvious—if a loading device is placed between two beams and the load increased, the beam having the lesser maximum resistance will fail. Since the load is limited to that required to fail this beam, the second beam can be subjected to no greater load and therefore could not fail, regardless of its energy absorption capacity.

For dynamic loads a similar situation is to be expected. A simple demonstration may illustrate this point. Consider the two resistance functions shown in Fig. C ($R_A < R_B$). If a step pulse as shown in Fig. D is considered to act equally on both beams, for any F equal to or greater than R_A but less than R_B there will be unlimited displacements associated with Beam

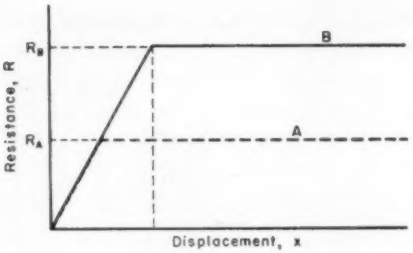


Fig. C—Two resistance functions, with $R_A < R_B$

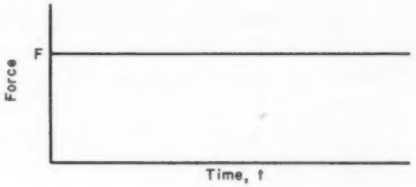


Fig. D—Step pulse shown is considered to act equally on both beams

A. For the same force F, Beam B will remain elastic or at most have limited plastic displacement depending on the relative values of R_A and R_B. No demonstration of relative resistances to dynamic loads would be provided under these conditions. Assuming that it remains elastic, Beam B could equally well have a resistance form shown by the light dashed line on Fig. C. If the area under the resistance curves is less for B than for A, Beam B would be less capable of resisting impulsive loads than Beam A.

The most satisfactory comparative demonstration would make use of an equal initial impulse to both beams. Since the area under the resistance curve is directly proportional to the initial impulse required for failure, the failure of one beam would demonstrate that it is weaker than the other when subjected to an initial impulse. If the results of this demonstration are the same as for static loading, then the results can be expected to hold for any type of dynamic load. If the comparative results differ for initial impulse and static loading, no general statements are possible, since the comparative strength depends on the loading. If neither beam fails, the relative maximum displacements give no indication of the relative resistance to impulsive loads, since the beam with the minimum displacement may be right at the collapse displacement while the other beam has substantial reserve strength.

The relative advantages or disadvantages of hard-grade reinforcing steel for resisting dynamic loads have not been established. The above paragraphs have attempted to show some of the reasons for this opinion and to suggest possible clarifications.

By F. WALLEY*

The work described in this paper is extremely interesting and adds to our knowledge of the way reinforced concrete beams behave under impulse loading. I do wonder, however, whether it proves that high tensile steel is better or worse than mild steel. What it does prove is that a beam which is stronger statically is stronger dynamically which for the type of loading employed is not altogether surprising. Surely what is important is to know how two

*Chief Structural Engineer's Branch, Ministry of Works, London, England.

beams, one with high tensile steel (which presumably would be smaller) and the other with mild steel, both designed for the same loads would behave. Additionally, since the type of loading is more akin to blast loading from high explosives, are the results quoted likely to be relevant to the blast loading from nuclear weapons where the duration of the pressure is relatively longer? Provision of stirruping is also considered an important factor in the design of beams for impulsive loading, but it does not appear to have been used in these tests.

I feel the matter is yet unproven one way or the other, but one is still left with a feeling that a bar with good ductility is better than one with small ductility.

AUTHORS' CLOSURE

The authors are grateful for the interest that has been shown by the discussers—pro and con—in this paper reporting test data on destructive impulse loading of reinforced concrete beams. The tenor of the discussions is for the most part by persuasion to a point of view rather than by presentation of additional test data which had not been reviewed by the authors. (One exception is a report cited by Mr. Fey which the authors have not seen.)

Mr. Balog concludes from the figures he quotes in his Table A (from Reference 83) "the superiority of the structural-grade steel St 37 over the high-strength steels in carrying impulse loads." Considering only the figures in his table the authors would draw a different conclusion. The significant comparisons would seem to be among figures in a given column rather than a given row. For example, the impulsive bending stress (Column 5) is about 1.5 times as high for Isteg and St 55 as for St 37; and about 1.9 times as high for St 80 as for St 37. However, a clearer picture can be obtained by plotting for each material the tensile yield stress as ordinate, and the static bending stress as abscissa, and opposite each point so plotted the value of the impulse bending stress. The four points so obtained from this Table A lie close to a parametric line defined by the equations

$$y = \frac{IB}{1.36} = TY$$

$$x = \frac{IB}{1.14} = SB$$

in which y is the ordinate; x is the abscissa; IB is the impulse bending stress; TY is the tensile yield stress; and SB is the static bending stress. The graph is not reproduced here because it does not have general significance—but this method of analyzing raw data graphically often has advantages over tabulating ratios which may obscure basic relationships.

Answering Mr. Balog's implied questions, there was no web steel in the test beams, and while the research suggested need for whiplash steel under

certain conditions, it was not within the scope of this study to determine the correct amount.

Mr. Helfgot asks what would be the relative amounts of different grades of steel for the same performance under impulse loads as stated in the paper. All beams tested by the authors were identical in size, and in area and location of steel. Accordingly, the authors' tests do not directly furnish data to answer his question. However, he might make his own inferences by studying the data quoted by Mr. Balog in Columns 2 and 5 of Table A. He might infer within the range of tests reported by the authors that if the steel governs one might expect about the same relative performance with structural-grade, intermediate-grade, or hard-grade steel if the design is based on yield strength of material—qualified by an appropriate judgment factor in design to account for variability of materials and service conditions. The tabulated data cited by Mr. Balog do *not* conflict with the results of the authors' experimental work.

Mr. McKee's mathematical treatment is interesting but the assumptions implicit in his Eq. (A) would make the equation invalid generally, and doubtfully accurate even if it were applied step-wise through small increments of time and space. The reasons are that in the authors' experiments

1. The resistance function was *not* independent of time.
2. The forcing function was spring-actuated and therefore *not* independent of displacement.
3. Damping is *not* negligible so that terms involving (dx/dt) would not be negligible.

The authors would not disparage mathematical analysis of the underlying problem with the objective of scientific clarification—but the *design* of slabs, beams, and columns for a structure bogs down if it becomes needlessly complicated.

Mr. Fey has been helpful in referring to the report by Galletly, Matsuda, and Ofjord which was not uncovered in the authors' exhaustive search of the literature—and they have not yet seen the report. Certainly there is need for additional experiments and analyses to bring a better working order out of the confusion which may for some time exist around the behavior of structures subjected to impulsive loads. In another paper* the authors have discussed certain other phases of the problem.

It would be interesting to set up an experimental program suggested by Mr. Fey's analysis leading up to Fig. B. Loads of shorter duration would be involved as he suggests, but it may not be easy to apply them by merely increasing the stiffness of a helical spring. A little figuring will show why. However, the loading problem can be handled by other means, and the instrumentation would be modified accordingly. The essential plans for such experiments have been worked out.

*Greaves, M. J., and Mavis, F. T., "Inelastic Behavior of Impulsively Loaded Beams," *Proceedings*, ASCE, Paper 1232, May 1957.

Mr. Hajnal-Kónyi suggested additional experiments in which the percentages of steel and the quality of concrete are varied under control. Subsequently, some work has been done on beams with different percentages of reinforcement and different grades of steel with beams loaded at the third-points by a different spring-loaded testing machine. That study has not yet been published.

Mr. Walley is left with a feeling "that a bar with good ductility is better than one with small ductility." If he means *enough* when he says "good" and *not enough* when he says "small" the authors are fully in agreement with him. However, in all tests reported in this paper, neither the hard-grade nor the intermediate-grade steel lacked sufficient ductility. The practical question may be raised whether excess ductility is frequently specified for reinforcing steel at the expense of other desirable properties.

Messrs. Hansen, Johnston, Newmark, and White have been intimately associated with problems of establishing design criteria with regard to loadings from high explosive bombs. Their second paragraph attributes to the authors a statement which they quote in part when in fact the authors quoted the statement from *The Effects of Atomic Weapons* (U. S. Government Printing Office, 1950), p. 380, as follows:

> "Tests have shown that in this respect [resistance of buildings to blast damage] a structural or intermediate-grade steel is much better for reinforcement than a hard-grade steel. When a reinforced concrete structure is subjected to heavy blast forces and damage occurs, the softer steel may elongate or deform without failure whereas the hard-grade steel snaps. More energy is absorbed in the plastic range by the structural or intermediate-grade steel. In the usual case such a selection will have little effect on rigidity."

In this paper the authors concentrated on a series of tests within (to be sure) a limited range of variables. Within this range not a single test showed the consequences generalized in the above quotation. They hope that the tests referred to in the quoted paragraph will be published and that in time engineers may have a better understanding of the behavior of reinforced concrete structures under impulsive loads.

Messrs. Hansen, Johnston, Newmark, and White state that "Obviously, tendencies toward brittle fracture [of steel bars in reinforced concrete beams (?)] were not even considered by the authors." Yet it should be evident that neither the authors' diligent study of an extensive literature (see bibliography of 116 entries) nor their own experiments have revealed any such "tendency" based on tests of reinforced concrete beams. If these "tendencies toward brittle fracture" of steel as reinforcement in concrete beams are so obvious, it would seem that pertinent test data should have assumed front-page prominence. Someone may have a ready opportunity to cite the significant published papers that were overlooked, or to publish a paper that will set the record straight.

Certainly "engineers must be careful about drawing sweeping conclusions from inadequate tests" and they should be equally cagey about accepting

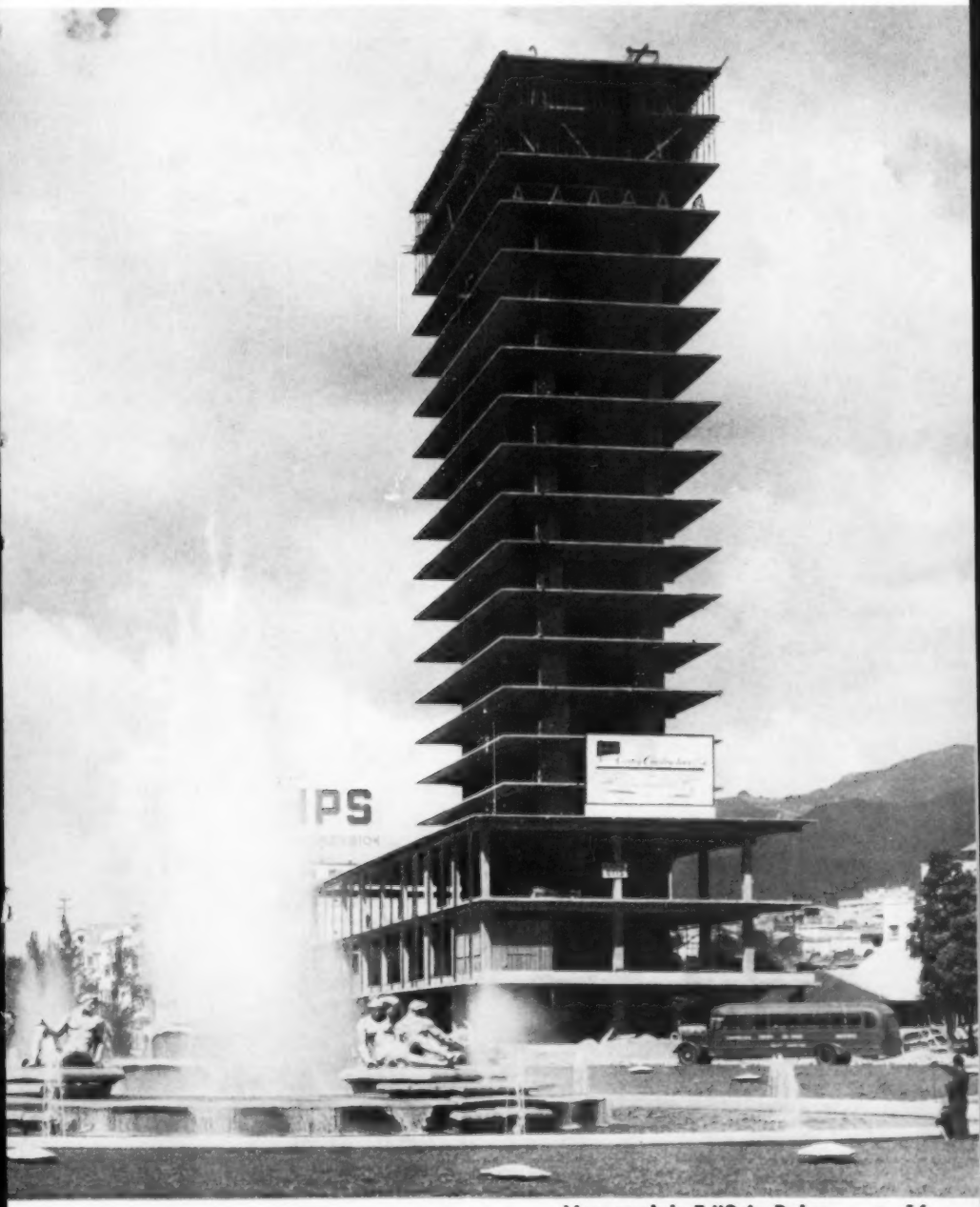
statements that "tests have shown" something, somewhere (if only someone could uncover the source).

The studies reported in this paper cover only a part of the broad field of the behavior of reinforced concrete beams under dynamic loads. Yet the part it does cover has been outlined as clearly as the authors could outline it, and recorded by high speed motion pictures for objective and detailed study. There are clearly enough "unknowns" about the general problem and how it can best be tackled in an engineering design office to keep experimenters, analysts, and engineers interested for a while. The fact that the paragraph quoted in this closing discussion has not been incorporated in the current edition of *The Effects of Nuclear Weapons* should not dull interest in the basic problem and its engineering applications.

The authors are grateful to all of the discussers for their constructive criticisms.

ACI News Letter

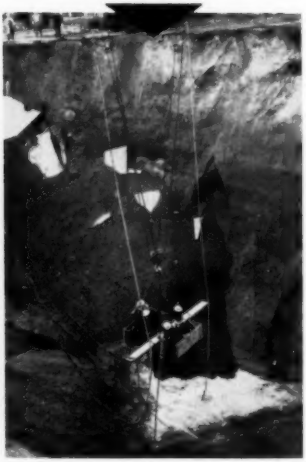
Vol. 29, No. 9 JOURNAL of the AMERICAN CONCRETE INSTITUTE March 1958



Venezuela's Edificio Polar . . . p. 16

CAVE-IN CALLS FOR QUICK ACTION

Seattle cave-in menaces entire residential area. At this point, hole measured 200 ft. long by 100 ft. wide.



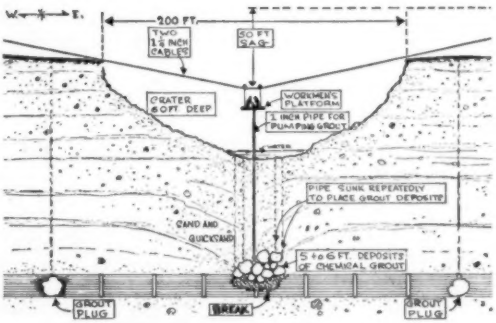
Working from swinging platform, Intrusion-Prepakt crews stabilize loose material by injecting chemical grout through pipes driven into the bottom of the hole. Flexible lines deliver grout from mixer and pump.

Emergency Grouting by Intrusion-Prepakt Halts Damage from Broken Sewer

Without warning, the city of Seattle was faced with an unexplained cave-in on Ravenna Boulevard. The size of the hole increased at an alarming rate despite the city's round-the-clock efforts to stop its progress. Shoring was attempted but failed when the 70 foot piling was swallowed up. Concrete pavement, sidewalks, trees and utility poles alike were consumed by the ever-widening crater. Homes in the vicinity were threatened, and a larger area suffered ruptured utility services.

The appearance of water in the 60 foot deep hole confirmed early fears of a collapsed trunk sewer, 140 feet below the street. Unable to cope with this creeping destruction, city authorities made a series of quick decisions. A general contractor was retained to repair the damaged sewer. The contractor called in INTRUSION-PREPAKT to plug the sewer with cement-sand grout and thus divert the flow into a hastily prepared by-pass. I-P crews then descended into the yawning pit on a cableway-supported platform, and from this precarious position, pressure grouted the loose material at the bottom of the hole with stabilizing chemicals. This halted further settling and caving, and permitted the hole to be filled to the original grade.

The Seattle cave-in is a dramatic example of INTRUSION-PREPAKT's rarely publicized ability to move in on emergencies—fast. When circumstances call for prompt, effective action, the resourcefulness and experience of I-P's world-wide organization are as close as your telephone. Immediately available is the same service that has given I-P recognized leadership in the fields of concrete construction, maintenance and planned stabilization programs. For information, contact INTRUSION-PREPAKT, INCORPORATED, 568-N Union Commerce Bldg., Cleveland 14, Ohio. In Canada: INTRUSION-PREPAKT LTD., 159 Bay Street, Toronto, Ontario.



Cross section illustrates how Intrusion grout plugs blocked-off sewer and permitted chemical grouting of loose material above break.

City Engineer—Roy W. Morse • General Contractor—MacRae Bros. Construction Co.

Photos courtesy The Seattle Times • Drawing courtesy Seattle Post-Intelligencer.

INTRUSION-PREPAKT, INC.

OFFICES IN PRINCIPAL U.S. AND FOREIGN CITIES

*Intrusion and Prepakt are trade names of Intrusion-Prepakt, Inc. whose methods and materials are covered by U. S. Patents Nos. 2313110, 2655004, 2434302 and others, also patents pending.

ACI

News Letter Contents

March 1958

On the Cover—Edificio Polar, Caracas, Venezuela, during construction. Four stark, tapering concrete columns pierce the cantilevered floor slabs to form an exciting backdrop for fountains and sculpture of the Plaza Venezuela. For a look at the finished building and the adjoining Teatro del Este, see p. 16.
—*Photo courtesy Cement and Concrete Association, London*

Honorary Members..... 3

Colorado Bridge Project..... 6

Powder Lance Cuts Concrete..... 9

Floodlighting Towers.....10

Positions and Projects.....13

Building Committee Report.....18

Who's Who
C. A. Vollick.....21
E. C. Wenger.....21
A. D. Ross.....21
Eugene Guillard.....21
John H. Doggett.....22

Looking Ahead.....22

Honor Roll.....27

New Members.....27

Tools, Materials, Services..31

List of Advertisers.....34

Freyssinet Parsons Walker

honorary members

announced during A C I convention

HONORARY MEMBERSHIP in the American Concrete Institute has been conferred on three eminent men in recognition of outstanding service in the field of concrete or in the work of the Institute. New honorary members are Eugene Freyssinet, Société Technique pour l'Utilisation de la Précontrainte, Paris; Douglas E. Parsons, chief, Building Technology Division, National Bureau of Standards, Washington, D. C.; and Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C. Their election was announced at the Awards Luncheon during ACI's 54th annual convention in Chicago, Feb. 26, 1958.

Since election of Richard L. Humphrey in 1926 as the Institute's first honorary member, 22 others have been similarly cited—a total of 26 honorary members in ACI history. Honorary members are elected by unanimous vote of the Institute's Board of Direction.

Eugene Freyssinet

Before 1923 prestressed concrete members were not sufficiently practical to compete economically or even technically with other structural forms. Researchers and engineers studying this form of construction had failed to recognize the inherent limitations of the materials. It was Eugene Freyssinet of France who recognized the importance of creep, the influence of which was not appreciated previously. His intensive studies of plastic flow and shrinkage in concrete demonstrated the necessity to use both high quality concrete and high-tensile steel. His basic patent was filed in France in 1928. A corresponding U. S. application was

Freyssinet

Parsons

Walker

filed in 1934, and since numerous subsequent patents covering his methods in beams and pipes have been filed in various countries. In 1939 he developed the method of anchoring post-tensioning cables by wedges using double-acting jacks.

Mr. Freyssinet was born in Objet (Correze) in 1879, and studied civil engineering at the Ecole Polytechnique where he specialized in bridges and structures. From 1905 to 1914 he was engineer of bridges and structures at Moulins and then became technical director of prestressed concrete development for Limousin and Co. until 1928. Since then he has engaged primarily in research and design in prestressed concrete.

An officer of the Legion of Honor, Mr. Freyssinet was awarded the Camere prize of the Society of Civil Engineers of France, the Elphege Baude prize of the Society for Encouragement of National Industries, and the gold medal of the same society of the National Office of Research and Inventions. In 1950 he received the Frank P. Brown Medal of the Franklin Institute. He was president of the Commission for Experimental Study of Concrete and Reinforced Concrete, and has been a member of many national and international committees and commissions on this subject. Mr. Freyssinet's work in concrete design can be seen throughout Europe. He was consulting engineer for the port of Stockholm during construction of the prestressed concrete bridge over the Traneberg Channel. His also was the design of the prestressed airstrip at Orly field near Paris. He now serves on ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

Douglas E. Parsons

Douglas E. Parsons, chief, Building Technology Division, National Bureau of Standards, has been active in ACI affairs since 1926. His record of achievement in the annals of Institute affairs presents an impressive picture. Culminating many years of activity with administrative and technical committees, as well as the Board of Direction, Mr. Parsons was vice-president in 1943 and 1944, and became president of the Institute in 1945.

In 1936 he was awarded the Wason Medal for Research for the paper "Tests of Mesnager Hinges" which he co-authored with A. H. Stang. In 1952 he received the Alfred E. Lindau award "for his many years of quiet perserverance in directing and coordinating development of standards of practice for reinforced concrete design." He has written a great number of papers in the Bureau of Standards series of publications, including more than a dozen of the Building Materials and Structures Reports on structural properties of various types of masonry construction. He also has been a frequent contributor to technical journals.

Mr. Parsons is currently a member of the executive group of Committee 115, Research;

he is a member of Committee 209, Volume Changes and Plastic Flow in Concrete; Committee 318, Standard Building Code; Committee 323, Prestressed Reinforced Concrete (joint ACI-ASCE); Committee 326, Shear and Diagonal Tension (joint ACI-ASCE); and Committee 335, Deflection of Concrete Building Structures.

He is a graduate of Cornell College, Mount Vernon, Iowa, and active in numerous professional societies. In 1919, following 20 months in the armed services, Mr. Parsons joined the engineering department of the Iowa Railway & Light Co., where he was employed until joining the National Bureau of Standards in 1923.

Full story of

A C I

54th annual

convention

held in Chicago, Feb. 24-27, 1958, will be carried in the April News Letter. Complete coverage of all technical sessions.

Stanton Walker

Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, is a man of outstanding accomplishments in the fields of concrete and aggregates, where he has been prominent in research, education, and the development of specifications and test methods.

A prolific writer, Mr. Walker has made significant contributions toward the advancement of concrete and aggregate technology in his many papers and reports to engineering organizations, many of which have appeared in the ACI JOURNAL.

A past president of ACI, Mr. Walker has been active in technical and administrative committee work of the Institute for many years. At the present time he is chairman of ACI Committee 605, Hot Weather Concreting and a member of the following ACI committees: 115, Research; 613, Recommended Practice for Proportioning Concrete Mixes; 617, Specifications and Recommended Practice for Concrete Pavements and Bases; and 621, Aggregates.

An engineering graduate of the University of Illinois in 1917, he was research engineer for Portland Cement Association from 1917 until 1926 when he assumed his present position with NSGA. In 1930 he also became engineering director for the ready-mixed concrete organization.

Among other honors accorded Mr. Walker are the ASTM Frank E. Richart award for outstanding service to ASTM and the Highway Research Board's Roy W. Crum Award for distinguished service. The Stanton Walker Fellowship has been established at the University of Maryland in his honor.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 321, Design of Reinforced Concrete Slabs (Joint ACI-ASCE)

Alan H. Mattock
Portland Cement Association
Chicago, Ill.

Committee 328, Limit Design (Joint ACI-ASCE)

D. T. Wright
Queen's University
Kingston, Ont.

Committee 331, Structures of Masonry Units

R. O. Hedstrom
Portland Cement Association
Chicago, Ill.

Committee 335, Deflection of Concrete Building Structures

George E. Large
Ohio State University
Columbus, Ohio

Colorado Bridge Project

112 post-tensioned girders used in construction of four bridges on Air Force Academy grounds

CONSTRUCTION OF FOUR highway bridges and two railroad bridges was recently completed on the grounds of the Air Force Academy north of Colorado Springs, Colo.

A total of 112 post-tensioned girders was employed in the highway bridges; each girder weighed 96 tons, was 120 ft long and had an over-all depth of 5 ft 11 in. Eight girders, spaced on 9-ft centers, comprised each span of the 88-ft divided highway.

A problem was confronted in positioning the prestressed concrete girders for the highway bridges following their on-site casting. Two 60-ton cranes which could have done the moving job were not readily available in the area. Use of four 30-ton cranes was ruled out because of the roughness of the terrain.

Ultimately, the following method was developed. Two gantries, each with a 50-ton chain block, were constructed. Mounted on rails which extended over the abutment on temporary steel trusses, these gantries, in effect, "straddled" each end of the girder in its casting bed. Subsequently, the gantry chain blocks hooked on to the lifting loops in each end of the girder and rolled it into its final position in the span (illustration above).

Each girder contained 12 Prescon post-tensioning assemblies consisting of 16–¼ in. diameter high tensile wires encased in flexible metal sheathing. After stressing to a total force of 1370 kips in each girder, through the use of power-driven portable 100-ton center hold rams, the metal sheathing was pumped full of grout.

After two spans of girders were erected, the diaphragms were formed and cast and the deck was concreted. The girders for the

remaining spans were rolled on rails placed on the existing deck and again across the temporary trusses.

Each of two railroad bridges contains two 72-ft spans. Four girders spaced at 5 ft 6 in. support the road bed. A total of 16 girders, each weighing 60 tons, were used in this phase of the project. The required final prestressing force of 1180 kips was applied through 11 tendons composed of 16–¼ in. diameter high tensile steel wires. Stress application, grouting procedures, and the cross section of the railroad girders were identical to that of the highway bridges.

Casting beds

Concrete casting beds were constructed on one of the approaches to each bridge. These casting beds were aligned with the final location of the girders on the piers and abutments. Wood side forms fabricated in 20 ft long panels were used. These panels were bolted to inserts in the concrete form soffit at the base and tied across the top. Each panel was used an average of 15 times before major repair was needed. Both internal and external vibration were used for consolidation. Each of the girders was fully prestressed and the tendons grouted before removal from the casting bed.

Concrete mix proportions

Specified 28-day cylinder strength of the concrete was 5500 psi, a cylinder strength of 4500 psi being required at transfer. This strength was achieved in an average of 4 to 5 days with no special curing.

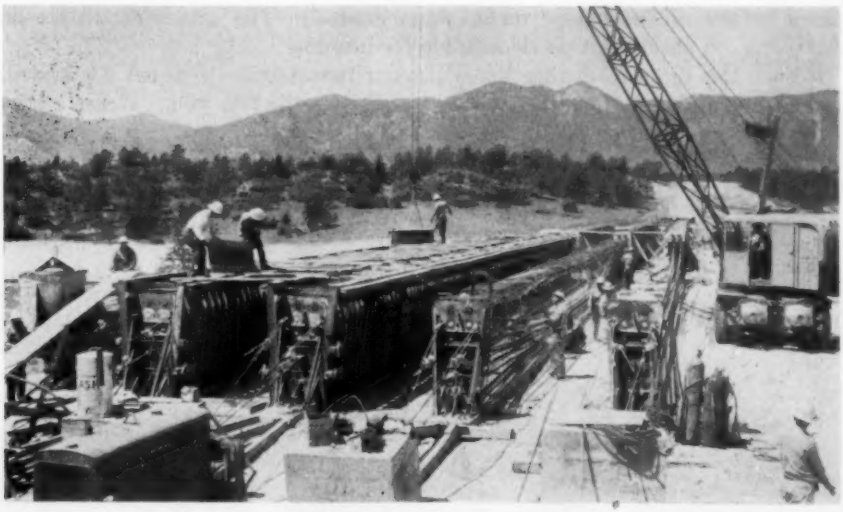
Concrete mix used for the girders was:

- 7½ sacks Type I cement
- 1300 lb sand
- 1760 lb coarse aggregate, maximum size ¾ in.
- 30.5 gal. of water (4.07 gal. per sack of cement)
- 1.88 lb Pozzolith and 4.1 percent air

The 2-in. average slump resulting from this mix produced an average 7-day strength of 6345 psi.

A. S. Horner Construction Co., Denver, was general contractor for this project which was completed last October. L. Boduroff of L. Boduroff and Associates, Denver, was consulting engineer to the general contractor. Skidmore, Owings, and Merrill, Chicago, were the designers. The Prescon Corp. provided tendons, stressing and grouting equipment, and supervised the placing and prestressing operations.

Preparing forms and post-tensioning assemblies for the highway and railroad bridge girders recently erected on the Air Force Academy grounds near Colorado Springs



This is to Certify that

John Smith

is a

Member

of this institute and fully entitled to the privileges granted by its Charter and Bylaws

_____
President

_____
Secretary-Treasurer

Date of membership: July 1, 1952

Above is shown the membership certificate now available to all interested members. (Student members become eligible for certificates when they advance to one of the higher membership grades.) The actual certificate is 10 x 12 in. on parchment paper suitable for framing.

Return this order form together with your remittance—to avoid the added expense of billing—and your certificate will be mailed to you. Please allow 90 days for preparation of the certificate.

Please accept my order for a membership certificate at the price of $2.00.

☐ Check ☐ M.O. Enclosed

My name is to appear exactly as follows:

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(Print or type)

Signature_____

powder lance cuts concrete

Powder lance in flaming action makes a vertical cut in 3 ft thick concrete wall

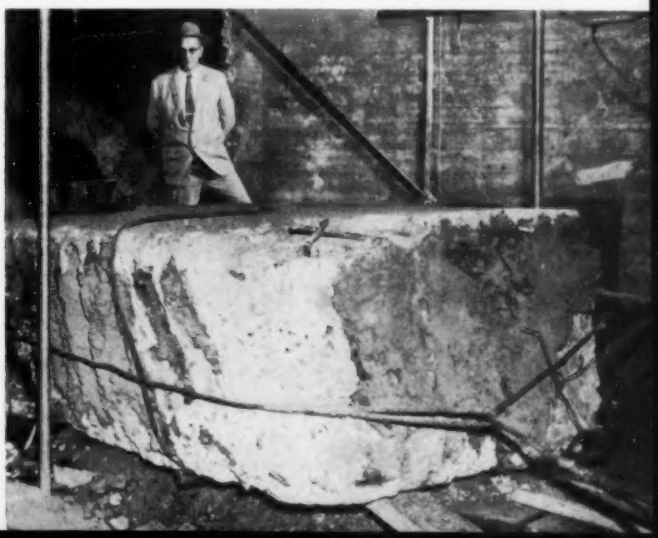
POWDER-LANCING has expedited a huge concrete demolition job for General Electric Co.'s general engineering laboratory at Schenectady, N. Y. Part of a laboratory building which formerly housed a high-speed pit having concrete safety walls up to 4 ft thick for testing steam turbines is now being converted into a new million-dollar radiation laboratory, scheduled for 1958 completion. Before construction commenced, however, the walls had to be modified; over 8000 cu ft of concrete was removed by powder-lancing, an outgrowth of the powder-cutting process introduced in 1943 by the Linde Co. as a method of cutting stainless steel.

Powder-lancing is widely used in the steel industry for producing shot holes in open-hearth furnace slag pockets. However, recent developments in apparatus and operating techniques have made it practical for cutting thick concrete—especially where mechanized demolition methods are too costly, where time is an important factor, and where blasting cannot be employed. With the powder lance, concrete may be demolished in huge sections, moderate sized boulders, or smaller pieces—as conditions require.

In powder-lancing, a mixture of iron and aluminum powder is fed pneumatically into a lance handle and mixed with oxygen. The lance itself is manually operated and consists of a lance handle with one or more lengths of black iron pipe attached. The powder and oxygen are mixed in the lance handle, carried to the cut by the pipe, and produce a bombardment of burning iron and aluminum particles which melts the concrete. Cuts in concrete 12 ft thick have been made with the lance; theoretically deeper ones are possible.

The turbine testing pit at General Electric's new radiation laboratory is shaped hexagonally and consists of two concrete safety walls separated by 6 ft of packed sand. The inner wall is 3 ft thick, the outer wall 4 ft thick. Both walls were about 16 ft high, and a large amount of this concrete had to be removed

Continued on p. 25



First massive section severed from pit's safety walls is checked by Richard Bartlett, who supervised demolition

Prestressing joins concrete pipe sections to form tall

FLOODLIGHTING TOWERS*

FOUR 106 FT HIGH CONCRETE TOWERS have recently been constructed to floodlight the new Merdeka Stadium at Kuala Lumpur, capital city of Malaya. Celebrations were held late last August at this stadium to mark the formal attainment of independence (*merdeka*) by Malaya.

The four towers are situated at the corners of the stadium. Actual height from ground surface to underside of the flood lighting platforms is 106 ft 6 in. Each tower consists of an upper portion fabricated from 42 in. internal diameter reinforced concrete pipes, with the lower portion or base consisting of 48 in. internal diameter pipes. Between the two different diameters there are junction rings or diaphrams which form the end anchorages for prestressing of the upper and lower portions.

Upper shaft

The upper shaft portion consists of 12 pipes 5 ft long, and the lower shaft is comprised of nine 5-ft lengths of pipe. Both pipe sizes had 2½ in. thick walls, and pipes were provided with double layers of helical reinforcement. The smaller pipes contained 32 ducts ¾ in. in diameter to accommodate the prestressing cables, and the 48 in. diameter pipes were provided with 36 ducts of the same size.

Prestressing

The towers were assembled and stressed on the ground, and ¾ in. thick cement composition joints were used between pipes. Each duct contained two 0.276 in. diameter high tensile wires and, after stressing, the ducts were pressure grouted with cement mortar to protect the wires from corrosion. The Gifford-Udall system of prestressing was adopted for this work.

*Condensed from an article which appeared in the Sept. 1957 issue of *Hume News* (Melbourne). Photographs courtesy Humes Ltd., Melbourne, Victoria, Australia.

Erection problems

The problem of erecting the high towers was difficult; in addition to the cumbersome 106-ft lengths to be handled, each tower weighed about 25 tons. Special lifting frames were therefore devised by the Malaya Public Works Department. The towers were first raised horizontally between the two lifting frames, then rotated in a vertical plane until the base of the tower was correctly centered. The towers were next plumbed vertically to the required accuracy and temporarily guyed while the lifting frames were dismantled.

Shaft bases in heavy foundation

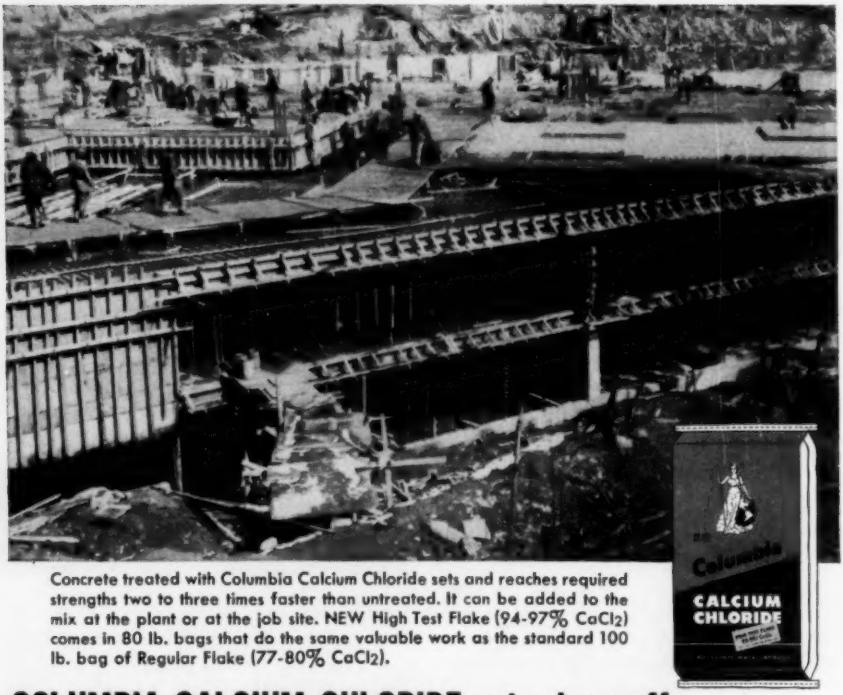
Each shaft base was set in a heavily reinforced concrete foundation, and buttresses were cast to transmit the bending moments for winds of 90 mph. Access to the floodlights is provided by means of steel ladders attached by cleats to the inside of the shafts.

The completed tower with lights in place is shown on the opposite page, with erection and assembly views on this page. Mortar which filled joints visible in the picture below was cured under wet burlap

The design of these towers was prepared by the Public Works Department, Federal Headquarters, Malaya, and the prestressing and erection was done by that department under the supervision of S. E. Jewkes. The reinforced concrete pipes for the project were supplied by Humes Industries (Far East) Ltd., Kuala Lumpur, Malaya.

Special lifting frame hoisted 106-ft concrete tower made of precast pipe lengths

Pipe sections were placed in position on an assembly base (picture below). In the spaces left between the pipes, ducts for prestressing wires may be clearly seen. After wires were threaded through the ducts, all the joints were filled with mortar



Concrete treated with Columbia Calcium Chloride sets and reaches required strengths two to three times faster than untreated. It can be added to the mix at the plant or at the job site. NEW High Test Flake (94-97% $CaCl_2$) comes in 80 lb. bags that do the same valuable work as the standard 100 lb. bag of Regular Flake (77-80% $CaCl_2$).

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More contractors this spring than ever before are saving days on their schedules. How? By using concrete ready mixes with Columbia Calcium Chloride added. Why? Look at what they get:

1. Faster initial set. Time is cut from 3 hours to 1 hour under normal conditions, more when the temperature drops below 70°F.
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4. Increased ultimate strength. Concrete cured with Columbia Calcium Chloride has 10 percent greater ultimate strength at the 5-year point.
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These all add up to savings on earlier form removal and less night finishing. Despite its superior resistance to low temperatures, however, Columbia Calcium Chloride is not an anti-freeze. Normal cold weather protective measures should always be followed. But the protection time will be reduced greatly—often from 7 days to 3—when Columbia Calcium Chloride is used.

For more information, write today to Calcium Chloride Dept. at our Pittsburgh address.

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Positions and Projects

ACI standards endorsed for Detroit area

The Concrete Improvement Board of Metropolitan Detroit at its first meeting for 1958 approved two ACI standards to "reasonably assure durable quality concrete."

The ACI standard "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" is recommended for selecting maximum water-cement ratio in terms of exposure conditions; "Recommended Practice for Winter Concreting (ACI 604-56)" also received CIB endorsement. In recommending ACI 604-56, CIB stipulated that at least minimum requirements under this standard be followed on semi-exposed placement, and that no less than the maximum requirements of this standard be specified for all exposed slab work such as sidewalks and driveways.

Willson re-elected to ASA Board of Review

C. A. Willson, research engineer, American Iron and Steel Institute, New York, has been re-elected to the Board of Review of the Standards Council of the American Standards Association. The Standards Council, judicial body of ASA, is responsible for the technical program of the association; it is made up of representatives of 65 national industrial, technical, and consumer organizations.

Mr. Willson, a member of ACI since 1936, is now secretary of ACI-ASCE Joint Committee 326, Shear and Diagonal Tension, and a member of ACI-ASCE Joint Committee 321, Design of Reinforced Concrete Slabs. He also serves on the following ACI committees: 115, Research; 208, Bond Stress; 209, Volume Changes and Plastic Flow in Concrete; and 318, Standard Building Code.

Richard H. Wilson retires

Completing nearly 34 years of service, Richard H. Wilson, assistant state highway engineer, California Division of Highways, has retired, turning his duties over to J. P. Murphy, planning engineer of the division.

University of California conducts construction conference

The third of a series of national conferences on construction operations planned by the engineering departments of the University of California and a distinguished advisory committee of engineers, educators, and construction industry leaders was held on the campus of the University of California, Los Angeles, Jan. 31 and Feb. 1, 1958.

Harmer E. Davis, chairman, division of civil engineering and irrigation, University of California, Berkeley, and D. Lee Narver, Holmes and Narver, Inc., Los Angeles, both ACI members, were among members of the advisory committee for the conference.

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February 1951 issue of

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Vol. XLVI, No. 2

Carbonell promoted by Sika

Celso A. Carbonell, manager in Panama City for Sika Chemical Corp., Passaic, N. J., for the past 5 years, has been promoted to executive vice-president and general manager of Sika Panama, S. A. He will be responsible for the operations of the company in Mexico, Central America, Cuba, Haiti, the Dominican Republic, British and Dutch possessions in the Caribbean, Venezuela, Peru, Ecuador, Bolivia, and Surinam.

Mr. Carbonell has been an active member of ACI since 1951. He now serves as a reviewer of Spanish bulletins on concrete technology and cement production for the "Current Reviews" section of the JOURNAL. ACI was represented by Mr. Carbonell at the Seventh Pan-American Highway Congress held in Panama City last year.

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EXECUTIVE HOUSE—combination hotel and apartment building at 63-73 E. Wacker Drive, Chicago
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Structural Engineer: Miller Engineering Company, Chicago

Another record year for
REINFORCED CONCRETE

1958 Promises to be another record year for reinforced concrete construction. A significant example of this trend toward reinforced concrete is the new 40-story structure shooting up on Chicago's Wacker Drive. It will be the tallest reinforced concrete structure in the U. S. A. Before you build, investigate this economical flexible, and time-saving medium of construction.

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Joseph M. Fink, center, receives the first annual Huron Achievement Award in Detroit: presentation is being made by Robert L. Freeman, Huron Portland Cement Co., and Dominic Rossi (right), new CIB chairman, looks on

CIB of Detroit honors Fink

Joseph M. Fink, chief of the bureau of structural engineering of Detroit's department of buildings and safety engineering, was unanimously chosen by the Concrete Improvement Board of Detroit to receive the first annual Huron Achievement Award for his outstanding contributions to the improvement of concrete in metropolitan Detroit area. Mr. Fink was cited particularly for his efforts toward adoption of the new Detroit building code, for developing recommendations for improved residential basements, and for "his continuing open-minded approach to establishing rules of safety for the use of new materials, designs, and methods of construction not contemplated, or provided for, in existing codes."

Concrete Improvement Board officers elected for 1958 were: chairman, Dominic Rossi, Darin and Armstrong, Inc.; secretary, Donald Ziegler, Cooper Supply Co.; and treasurer, Thomas Burke, C. J. Burke Co.

PCA installs "electronic brain"

The Portland Cement Association has put into operation a two-unit electronic computer at its Chicago headquarters. The "electronic brain" is being used to prepare programs for design of concrete pavements, structures, and materials for design manuals. These will be made available to engineers and architects, enabling them to save design time and manpower.

The machine, a Bendix G-15 D general purpose digital computer and a DA-1 differential analyzer, has already been used to prepare design data on dowels for pavement, barrel shell roofs, and box culverts. Programs are also being developed for design of prestressed bridges, using both the standard sections developed by the Bureau of Public Roads and American Association of State Highway Officials.

Computer programs will be distributed through the Bureau of Public Roads, the Bendix Users' Conference, and other groups; some will also be available on request.

Sprague transfers for tour of service

John C. Sprague, formerly director, South Atlantic Division Laboratory, Corps of Engineers, U. S. Army, Atlanta, has transferred for a tour of service as chief, Foundation and Materials Branch, U. S. Army Engineer District, Gulf, in Iran.

Mr. Sprague has been an ACI member since 1931 and is the author of numerous technical papers, many of them published in the ACI JOURNAL.

Courtesy Cement and Concrete Association

Cover girl in full dress...

EDIFICIO POLAR, Caracas, shown during construction on this month's News Letter cover, appears here fully clothed in its skin of glass and insulated aluminum. Overlooking the University City and adjacent to a main traffic center, the vertical office tower rises from a horizontal block, and is linked with the Teatro del Este seen on the right.

Four concrete columns pierce the low building and taper through 15 upper floors. The solid floor slabs cantilever from these inner supports, with sloping cantilevered beams projecting from the four main columns beneath each floor. Where the vertical block leaves the horizontal, an open floor emphasizes the tower's slender construction.

External walls are of glass and insulated aluminum, with the metal faced with plywood on the inside. Continuous vertical steel I-sections fixed at floor edges hold the panels which are subdivided by T-sections and angles.

The air-conditioned theater, only fully equipped one in Venezuela, is boldly framed with exposed prestressed concrete. Frames are of a simple "goal post" form, spanning across the fan-shaped auditorium. The concrete slab roof is topped with asphalt, and painted with aluminum. Patterned brickwork carried on tie beams between the main frames forms the curtain walls. Concrete frame members were brush hammered, the bricks sprayed with varnish.

Theater balcony cantilevers from concrete frames shaped as inverted triangles with their apexes resting on the ground floor, and is supported at its upper edge by free-standing columns from basement and foyer. Ducts and service and projection rooms are housed in a superstructure of concrete ribs joined to the upper balcony construction and the last main auditorium frame. Entrance foyers, stairs, and access galleries are placed under the balcony, visible from outside through clear glass walls. At every point structure governs design, both internally and externally.

Architects for both Edificio Polar and Teatro del Este were Vegas and Galia; engineers, Tecnica Constructora C. A.

CRL to include concrete laboratories in inspections

The Cement Reference Laboratory as of Mar. 1, 1958, will include the inspection of laboratories which test concrete as well as the cement laboratories, according to announcement in the February *ASTM Bulletin*. Inspection will be undertaken on the voluntary request of the laboratory to be visited.

CRL, since its founding in 1929, has been inspecting laboratories of cement producers and major consumers, and independent laboratories for the purpose of giving freely requested assistance and instruction in the correct testing of cements. It operates from headquarters at the National Bureau of Standards, Washington, D. C., and is supported jointly by the Bureau of Standards, the American Society for Testing Materials, the Bureau of Public Roads, and the U. S. Army Corps of Engineers.

J. R. Dise heads the Cement Reference Laboratory, and A. Allan Bates, Portland Cement Association, Chicago, is chairman of the Cement Reference Laboratory Subcommittee of ASTM.

Lerner returns to Brown University

Samuel Lerner, professor at Brown University, Providence, R. I., who was on sabbatical leave during 1956-1957, has returned to Brown to resume teaching duties. Professor Lerner spent the year in Italy at the University of Turin studying structural engineering and was awarded the Doctor of Engineering degree in July 1957.

Chain Belt announces executive appointments

Election of an executive vice-president and three new vice-presidents at Chain Belt Co., Milwaukee, has been announced by L. B. McKnight, president.

O. W. Carpenter, formerly vice-president in charge of construction machinery and finance, is now executive vice-president, with direct responsibility for current operations of the company.

Newly elected vice-presidents include W. C. Messinger, construction machinery section; E. M. Rhodes, industrial equipment section, and G. H. Woodland, marketing.

Rahn retires

G. A. Rahn, formerly materials engineer with the Pennsylvania Turnpike Commission, Harrisburg, recently retired from service.

University of Wisconsin concrete conference

An Engineering Institute on ready-mixed concrete technical problems was attended by some 75 persons at the University of Wisconsin, Jan. 15-17, 1958. The meeting was sponsored by the University Extension Division, and the program was organized in cooperation with ACI, the Ready Mixed Concrete Association of Wisconsin, Portland Cement Association, and the National Ready Mixed Concrete Association.

Among ACI members participating in the program were: M. O. Withey, dean emeritus of the college of engineering at the University of Wisconsin, who gave the welcome address; W. D. Kimmel, PCA, Milwaukee, who spoke on quality concrete, basing his talk on PCA's long-time study; and George W. Washa, University of Wisconsin, who spoke on "Basic Factors in Concrete."

Other Institute members participating included Paul G. Fluck, University of Wisconsin, who covered aggregate and concrete tests; Paul F. Rice, ACI technical director, spoke on "Specifications, Inspection, and Testing of Concrete." John Banker, PCA, Chicago, discussed experience with ready-mixed concrete.

Delmar L. Bloem, National Ready Mixed Concrete Association, opened the January 17 session with a talk on production problems of ready-mixed concrete. Later in the day at a session devoted to equipment of the ready-mixed concrete industry, B. C. Froemming, Central Ready Mixed Concrete Co., Milwaukee, discussed trucks for ready-mixed concrete. W. S. Cottingham, University of Wisconsin, presented certificates to those in attendance prior to adjournment of the conference.

TWO MILLION GALLON ELEVATED TANK nears completion in Tyler, Tex. Tank is elevated on 160 ft tower; 195 ft to overflow. Tank core wall is finished, and upon roof completion the tank wall will be prestressed by wire winding machine. Tank design is by The Preload Co., Inc., New York. Whittle Contracting Co. began work on the job in August 1957. Wisenbaker, Fix, and Associates are consulting engineers for the City of Tyler. Construction is under the direction of Clarence P. Stewart, city manager, J. P. Ferrill, city engineer, and Buck Campbell, water superintendent, all of the city of Tyler.

Building industry of California honors Streblow

A. G. Streblow, president of Basalt Rock Co., Napa, Calif., was recently presented the Achievement Award of 1957 by the Building Industry Conference Board at their annual awards dinner in San Francisco. He was recognized for his "ingenuity and creative ability" and his pioneering efforts in producing prestressed concrete masonry panels and precast, prestressed lightweight concrete structural members.

Mr. Streblow is a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete.

Needles takes office as president of EJC

Enoch R. Needles, senior partner in Howard, Needles, Tammen & Bergendoff, New York, recently assumed presidency of the Engineers Joint Council. Mr. Needles was president of the American Society of Civil Engineers in 1955-56, and has been an ACI member since 1948.

Building Committee Report

as of January 31, 1958

Most recent report of the ACI Building Committee shows the following contributions to the Building Fund, arranged by categories.

Cement producers	$ 76,802.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,206.22
Contractors	19,445.00
Ready-mixed concrete and aggregate industry	12,548.87
Membership at large	9,048.69
Admixtures manufacturers	7,526.83
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,595.00
Special contributions	1,766.43
Eastern Canada	1,496.90
	$ 209,020.94
Pledges in hand	2,760.00
GRAND TOTAL	$ 211,780.94

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Weather conditioning under canvas

Tents constructed of 20 x 20-ft canvas tarpaulins supported on a timber skeleton arose within a forest of concrete columns to expedite construction of the International Business Machine Corp. electric typewriter plant at Lexington, Ky. Heated air was blown into the canvas enclosures (blower at center of picture) to maintain 70 F curing temperature for post-tensioned beams and girders. The shelter also made humidity control possible during the 7-day curing cycle. These structural members range in length up to 69 ft, some weighing as much as 20 tons.

The single-story IBM plant, of precast, post-tensioned, rigid frame construction is made up of 40 x 50 ft bays, and more than 20,000 cu yd of concrete was placed in the prestressed members. Prime contractor for the job, Gilbane Building Co., Providence, R. I., selected tarpaulins manufactured by Hoosier Tarpaulin and Canvas Goods Co., Indianapolis. Similar heated enclosures of canvas were also used to facilitate floor slab curing.

Mardulier named manager of cement mill products

Francis J. Mardulier has been appointed manager of cement mill products, Dewey and Almy Chemical Co., Cambridge, Mass. Mr. Mardulier, whose responsibilities formerly were divided between cement mill products and the firm's construction specialty chemicals, will devote full time to cement mill products.

A graduate of the Harvard engineering school, Mr. Mardulier was formerly an instructor there, and was more recently a guest lecturer in cement and concrete technology at the Harvard summer school and Northeastern University. An ACI member, Mr. Mardulier is currently a subcommittee chairman of ASTM Committee C-1, and a member of the Harvard Engineering Society.

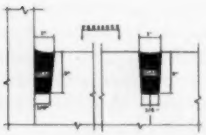
Sika staff enlarged

New additions to staff of Sika Chemical Corp., Passaic, N. J., include Thomas Seale who will open a district office in Atlanta, and Harold Brown who will open another Sika district office in New Orleans. George Breen will join the New York sales staff, and John Legrand will join the Dallas division staff.

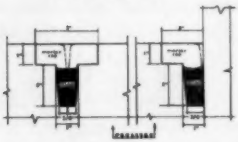
Oliver retires from Raymond Concrete Pile

Herbert Dean Oliver has retired as district manager of Raymond Concrete Pile Co., Atlanta office, after over 38 years in that position. He has been an ACI member since 1927. Herschel B. Miller, Houston district manager since 1951, will take over Mr. Oliver's post in Atlanta.

MAKE WATERTIGHT JOINTS with IGAS



Water Reservoirs
Tanks
Parking Decks



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Igas Joint Sealer is a non-meltable, mastic waterstop that is installed in joint slots after concrete has hardened. Igas will not dry out or become brittle when subjected to climatic changes . . . Because Igas is installed after concrete is placed, concreting operations cannot damage the waterstop . . . Igas is used in vertical, horizontal and overhead joints — thus forming a continuous watertight seal at complicated joint intersections . . . Igas is ideal for water reservoirs, sewage plants, swimming pools, foundations, basements, tunnels and similar structures . . . Igas has been used for more than twenty-five years . . . For complete information about Igas Joint Sealer, write for Bulletin IS-56.

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Who's Who This Month

C. A. Vollick

"Effects of Revibrating Concrete," appearing on p. 721, has been submitted by C. A. Vollick, who heads the research laboratory of Sika Chemical Corp., Passaic, N. J.

Following graduation from the University of Denver in 1938, Mr. Vollick worked for the U. S. Army Corps of Engineers at Lowry Field where he organized a field laboratory to test concrete and soil during construction. In 1942 he transferred to the U. S. Bureau of Reclamation where he worked in the concrete laboratory, in charge of a section responsible for concrete aggregate and riprap.

He returned to the Corps of Engineers in 1955 and was head of a section responsible for surveys, soil inspection, and concrete inspection for the Nike sites constructed in the Chicago area. He left this post to assume his present position with Sika. Mr. Vollick has been an Institute member since 1946.

E. C. Wenger

"Concrete for Sewage Works," p. 733, was prepared by E. C. Wenger, now with J. E. Evans Concrete Products Co., Indianapolis. Mr. Wenger retired last December as manager of the Conservation Bureau of the Portland Cement Association, a post he had held since 1947. Mr. Wenger had been a member of the PCA staff since 1934; for 13 years he was regional highway engineer in the association's midwest region.

Previously he had served the Illinois State Highway Department for 6 years as junior and district engineer.

Mr. Wenger received his engineering degree from the State University of Iowa in 1914. He worked for the Kansas State Highway Department for 2 years and later was maintenance and supervising engineer in the Cook County Illinois Highway Department for 4 years. From 1926 to 1932 Mr. Wenger was chief engineer for an Illinois firm of consulting engineers in the municipal improvement field. He became an ACI member in 1946.

A. D. Ross

A. D. Ross, professor of civil engineering, King's College, University of London, contributed "Creep of Concrete Under Variable Stress," appearing on p. 739 of this month's JOURNAL.

Professor Ross graduated in civil engineering from Edinburgh University, Scotland. After several years of experience in railway and road construction, he returned to Edinburgh to undertake a program of research in concrete technology for which he was awarded the PhD degree. After a period of service as education officer with the Air Ministry, he went to Kings College as lecturer in civil engineering, assuming his present duties there in 1946.

His numerous published papers describe research on the creep of concrete and its effects on structures. He has also devoted much time to thermal studies in mass concrete and in steam cured products, as well as the development of prestressed concrete. Professor Ross is a Fellow of the Royal Society of Edinburgh and a member of the Institution of Civil Engineers (England). He serves on the editorial board of the *Magazine* of *Concrete Research*, and on various committees of the Department of Scientific and Industrial Research.

Eugene Guillard

A study on the "Ultimate Resisting Moment of Beams with Compression Reinforcement" by Eugene Guillard, research engineer in the structural analysis section at Armour Research Foundation, Illinois Institute of Technology, Chicago, appears on p. 759.

Since graduation from the University of Illinois in 1938, he has been engaged in the design of railroad and highway bridges for the Illinois Central Railroad Co., copper mill buildings, and other structures for the Nevada Consolidated Copper Corp., refinery structures and equipment for the Standard Oil Co. (Indiana), and public works facilities for several consulting engineering firms. Prior

to joining the staff at Armour Research Foundation in 1951 Mr. Guillard taught structural engineering at the University of Texas and Tri-State College.

His recent research has been on the ultimate strength of structures, structural dynamics, and weapons effects. Most of this work has been in connection with the analysis and design of structures subjected to blast loading from nuclear detonations.

John H. Doggett

The construction paper on "First Slip-Formed Apartment Building in the United States," appearing on p. 767, was prepared by John H. Doggett, architect, Memphis, Tenn. This paper is based on a report originally presented at the ACI regional meeting in Atlanta, Nov. 1, 1955. Mr. Doggett tells the story of formwork and concreting for the upper eight stories of the ten-story Holiday Towers completed in Memphis in February, 1956.

Mr. Doggett graduated from Freed-Hardeman College, Henderson, Tenn., in 1919. After 24 years of varied architectural experience with private and governmental agencies, Mr. Doggett organized his present firm whose principal works have included hotels, office buildings, and college units throughout the country.

BRAB-BRI hold joint meeting

The National Academy of Sciences' Building Research Advisory Board and the board of governors of the Building Research Institute conducted a joint meeting February 11-12.

E. X. Tuttle, Giffels & Rosetti, Detroit, presided as chairman of BRAB. Among 12 of the nation's foremost authorities analyzing problems confronting the construction industry were ACI members: Charles H. Topping, BRI president, of E. I. du Pont de Nemours & Co., Wilmington, Del.; E. K. Abberly, Turner Construction Co., New York; A. Allan Bates, Portland Cement Association, Chicago; and Leonard G. Haeger, Levitt & Sons, Levittown, Pa.

LOOKING AHEAD

Mar. 12-15, 1958—American Concrete Pipe Association, Fiftieth Anniversary Convention, Roosevelt Hotel, New Orleans, La.

Apr. 21-22, 1958—Building Research Institute, Seventh Annual Meeting, Shoreham Hotel, Washington, D. C.

May 5-10, 1958—Federation Internationale de la Precontrainte, 3rd International Congress on Prestressed Concrete, Benjamin Franklin Congress Hall, Berlin, Germany.

May 20-21, 1958—Society of American Military Engineers, 38th Annual Meeting, Mayflower Hotel, Washington, D. C.

Rowe joins Ammann & Whitney

ACI member Stuart Rowe, formerly in the process plants division of Foster Wheeler Corp., New York, is now associated with Ammann & Whitney, New York.

Jelley opens civil engineering office

Rear Admiral Joseph F. Jelley has opened an office for the practice of civil engineering in Colorado Springs, Colo. Admiral Jelley, who recently retired from the U. S. Navy Civil Engineer Corps after 30 years of service, was chief of the Bureau of Yards and Docks during the Korean War. He later served as director of construction for the Department of Defense. His most recent assignment was with the Tenth Seabee Brigade with headquarters at Pearl Harbor. Admiral Jelley is a member of the American Concrete Institute, which he served as a director from 1954 through 1956. He is also a past president of the Society of American Military Engineers.

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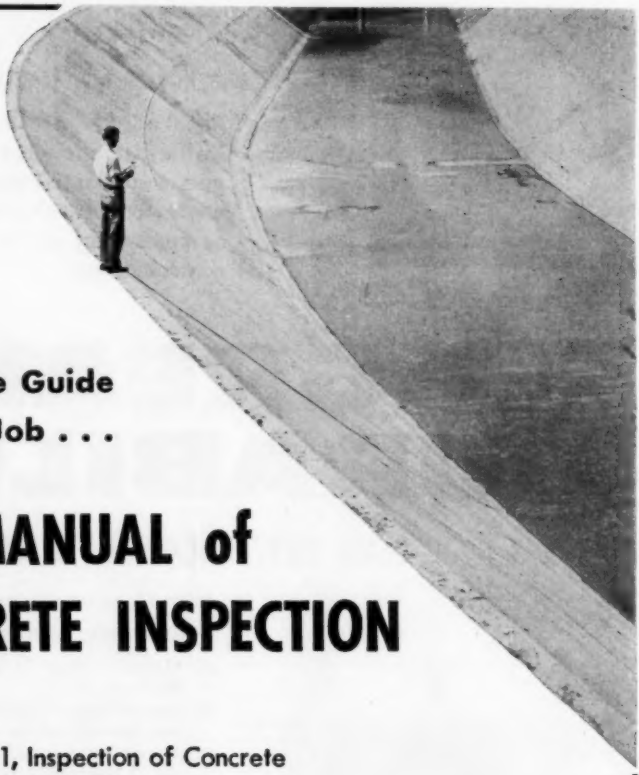

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Ackerly speaks at Construction Specifications Institute

Morton E. Ackerly, professional specifications writer, New Brighton, Pa., addressed the January meeting of the Pittsburgh Chapter of the Construction Specifications Institute. Mr. Ackerly defined building specifications and outlined seven basic steps for writing accurate and understandable specifications. (1) preview the project in an outline specification; (2) collect and correlate the source of information for materials and systems to be used in the project; (3) formulate the rough draft of the specifications; (4) check and correct the rough draft against finished contract drawings, other trades; (5)assist in compilation of contract provisions; (6) reproduce the specifications; and (7) test your specifications.

Structural concrete conference at University of Missouri

A structural concrete conference will be held Apr. 11-12, 1958 at the University of Missouri, Columbia. The conference will feature structural design problems in prestressed concrete, ultimate strength design, and thin shells and folded plates.

Speakers will include William Dean, Florida State Highway Department; Leo Corning, Portland Cement Association, Chicago; ACI past-president Frank Kerekes, Michigan College of Mining and Technology; Ivan Viest, University of Illinois, and Milo S. Ketchum, Ketchum and Konkel, Denver. The conference is co-sponsored by PCA and the university in cooperation with ACI.

Klock retires from Eastman Kodak

Morgan B. Klock, structural engineering consultant, representative to ACI for Eastman Kodak Co., Rochester, N.Y. (corporation member of the Institute), has retired from the company. Robert R. Sheridan, structural engineering consultant, will assume his representation in ACI for Kodak.

Mr. Klock has been actively interested in Institute technical committees for many years and presently is a member of ACI Committee 216, Fireproofing or Fire Protection of Structures.

Powder lance . . .

Continued from p. 9

to meet the construction specifications for the laboratory.

The concrete walls were cut by powder lance into sections measuring 20 x 16 ft, weighing up to 18 tons each. Cutting speed was about 1½ ft per hr. After each section was severed, it was hoisted out of the pit by 100-ton bridge crane and hauled away.

This unusual demolition project was handled by James E. Lowe & Sons of Schenectady. H. N. Lowe, president of the firm, estimated that the lancing operation would consume 20,000 lb of iron powder, 6000 lb of aluminum powder, 8000 ft of black iron pipe, and 450,000 cu ft of oxygen.

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Honor Roll

January 1—February 28, 1958

Blas Lamberti remains in first place on the Honor Roll this month with 7 credits.

ACI offers up-to-the-minute "know-how" on problems encountered in the field of concrete, and ACI membership is, therefore, essential to professional advancement.

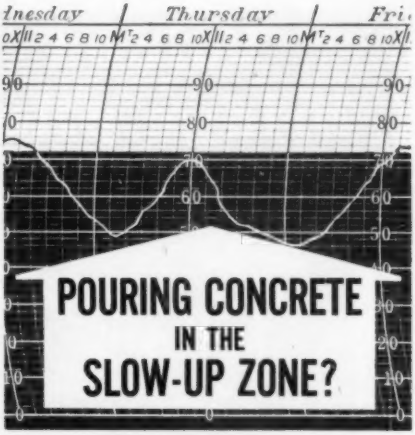
Blas Lamberti	7
Ernst Maag	4
Ernest L. Spencer	4
Joseph J. Waddell	4
Celso A. Carbonell	3
Phil M. Ferguson	3
Samuel Hobbs	3
Miles N. Clair	2
Jaime de las Casas	2
Carl O. Knop	2
George Kurio	2
James R. Libby	2
William McGuire	2
Douglas McHenry	2
H. C. Pfannkuche	2
J. P. Thompson	2
Ellis S. Vieser	2

New Members

The Board of Direction approved 97 Individual applications, 3 Corporations, 9 Juniors, and 18 Students, making a total of 127 new members. Considering losses due to resignations, deaths, and nonpayment of dues the total membership on February 1 was 9274.

Individual

AMPOLLINI, PHILIP P., Belvidere, Ill. (Proj. Mgr. Gannett, Fleming, Corddry & Carpenter, Inc.)
BAER, W. STANFORD, York, Pa. (Cons. Struct. Engr.)
BARBER, IRA E., Kansas City, Mo. (Supt., Ash Grove Lime & Portland Cement Co.)
BECK, CHARLES C., Elgin, Ill. (Mgr., Elgin Br., Pittsburgh Testing Lab.)
BEEDE, KENNETH ALLEN, JR., Emeryville, Calif. (Test Engr., Pacific Gas & Electric Co.)
BOOTHE, RAYMOND H. F., Los Angeles, Calif. (Struct. Research Engr., Holmes, & Narver, Inc.)
BOURKE, MILTON F., Concord, Calif. (Struct Engr., U. S. Steel)
BRENNAN, GEORGE P., Riverside, Calif. (Chf. Field Design Office, Kaiser Engrs., Inc.)
BRICKHAM, LEONARD A., Los Angeles, Calif. (Struct. Design Engr., Corps of Engrs., U. S. Army)
BROOKS, EDWARD T., Orange, N. J. (Prin. Engr., Louis Berger & Assocs.)
BROWN, CLAUDE M., JR., Toledo, Ohio (Struct. Field Engr., PCA)
BROWN, LEVI S., Skokie, Ill. (Research Petrographer, PCA)
BURNS, ALBERT V., JR., Callao, Peru (Chf. Matls. Engr., Utah Constr. Co.)
CANSINO, ANTONIO R., Oakland, Calif. (C. E., Supv. Constr., Stauffer Chemical Co.)
CARRASQUILLA Q., Uriel, Managua, Nicaragua (Dr. in Science of Civil Engrg., Cardenal-Lacayo-Fiallos, Archs, Engrs., Contrs.)



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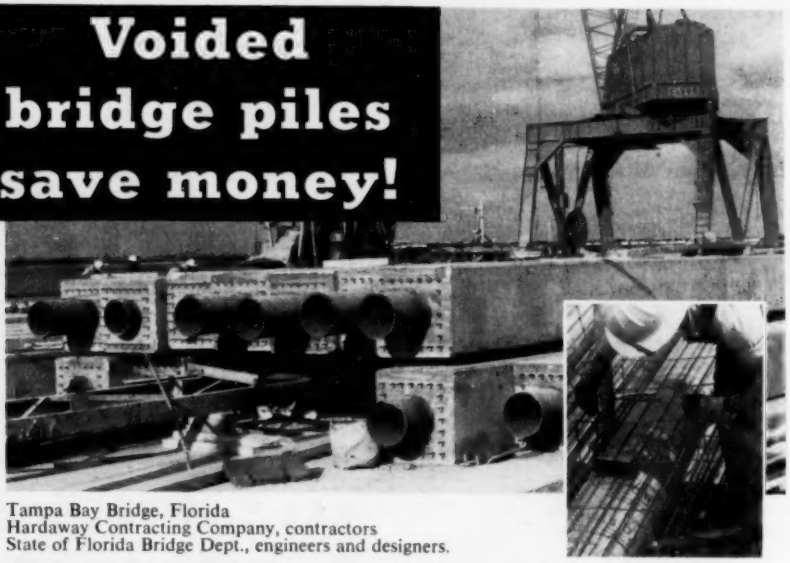

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State of Florida Bridge Dept., engineers and designers.

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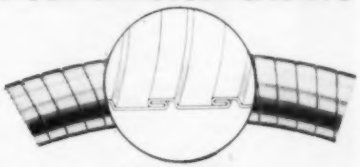

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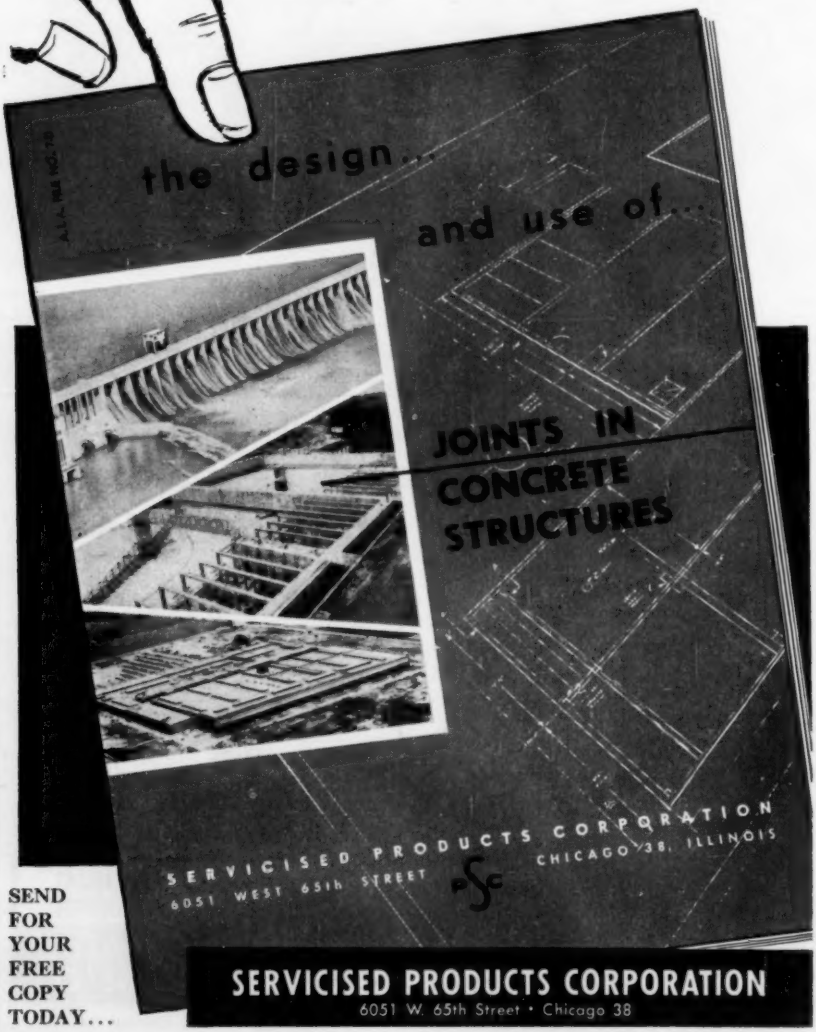

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Non-tilting concrete mixer

Non-tilting concrete mixer has drum 42 in. in diameter by 33 in. wide; the capacity is 6 cu ft of mixed concrete plus 10 percent overload. Mixer has channel type drum track; drum rollers and countershaft bearings are "Sealmaster." The discharge opening is 17 in. and the one-piece discharge chute is 27 in. high to facilitate loading into wheelbarrows. The mixer has a Wisconsin engine, rated at 10 hp at 1600 rpm, with a built-in clutch with chain drive to the countershaft. Disc-type wheels are Timken-equipped and the axle is mounted on coil springs. Tires are 5.90 x 15. The calibrated syphon-type water tank has a capacity of 10 gal. Bulletin A-36 contains complete details.—Muller Machinery Co., Inc., Metuchen, N. J.

Prestressing aids

"Preformed Steel Splices and Dead-Ends for Prestressed Concrete Tensioning Elements," a 14-page technical report by Howard F. Stirn, is now available. Splices and dead-ends preformed of high-strength uncoated or galvanized steel wire are available for commonly used sizes of prestressing wire and seven-wire strand. The report pictures assembly and installation, gives tensile and elongation test data for both splices and dead-ends.

Two types of dead ends are available: The "full tension" design is made of the same sizes and types of wire as the equivalent size splice. The "limted tension" unit is designed to hold not only the required prestress tensioning load but also to have strength at least equivalent to commonly used wedge-type connections. These preformed assemblies are wrapped in "vapor phase inhibitor" paper and shipped in cardboard boxes; no oil application is required to prevent corrosion.—Preformed Line Products Co., 5349 St. Clair Ave, Cleveland 3, Ohio

Waterproofing, repair and restoration compound

An epoxy-based polyplastic alloy is reported suitable for waterproofing, skidproofing, patching, and joining concrete and concrete masonry surfaces. Permagile "S" is a syrupy, self-leveling brush-consistency material, neutral in color, which may be used alone or mixed with fine aggregate to form mortar. Permagile No. 10, also neutral color, is heavier, viscous and non-sagging, intended for vertical and overhead work. A third form of the product is liquid, applicable by brush, roller, or spray, and available in colors.

Surfaces to which the compound is applied must be free of grease, oil, dirt, loose particles, efflorescence, and

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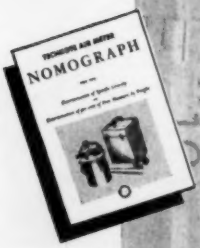

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all moisture; temperature should be 55 F or above. For use on moist surfaces, a primer and special cladding formulation are also available.

Permagile will harden within 8 to 12 hr to a reported average compressive strength of 35,000 psi, and average tensile strength of 9000 psi. Manufacturer reports that it is completely water resistant up to a pressure of 4320 psf. One pound of the compound will cover approximately 20 sq ft of dense masonry surface, according to producer's specifications.—Permagile Corp. of America, 34-43 Fifty-Sixth St., Woodside 77, N. Y.

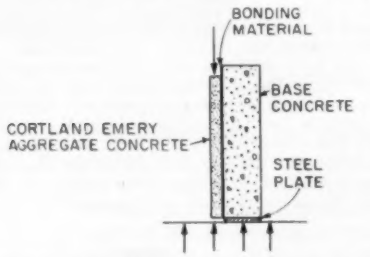
Transfer aids for concrete detailing

Press-on transfers to save drafting time are available in sheets containing general structural notes for reinforced concrete, with reference to ACI Standards; e.g., "Follow ACI 318 and ACI 315 for details and general procedure." Another sheet contains symbols and abbreviations commonly used in concrete detailing. Lettering is vertical capitals (plain semi-bold Gothic), $\frac{1}{8}$ in. high. The transparent supporting film is Mylar, reportedly tough and dimensionally stable.

Any set of notes or data can be made to order in these transfer sheets, which are to be applied to the under side of the master drawing.—Garrett Becker, Jeremiah Road, P. O. Box 535, Sandy Hook, Conn.

Concrete floor repair material

Tests were conducted by Haller Testing Laboratories, Plainfield, N. J., on Emeri-Bond, a non-reversing neoprene bonding material, designed to hold heavy duty Cortland emery aggregate concrete resurfacing material to an old concrete floor. Special concrete slabs were topped with the bonding material and a



¾-in. layer of Cortland emery aggregate concrete was applied to simulate an actual heavy-duty patch. The test samples were cured 30 days before testing.

Tensile strength tests caused failure of the standard test rigs before any discernible effect on the bond took place. The shear test, illustrated above, showed a resistance of 233 psi; when rupture occurred, it was through the bonding material itself.

For the impact tests, weights up to almost 8 lb, dropped from a height of 10 ft 7 in., caused no damage to the bond, and produced only light indentation or chipping on the surface of the Emeri-Crete flooring. The final object dropped, a 31.8-lb steel cylinder, on the first drop produced a crack in the test slab, but did not break it completely. The second drop produced another crack, which permitted the entire unreinforced specimen to be pulled apart.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

Hot pour joint sealer

Model AC-40A hot pour joint sealer is equipped with a material pump called the "Moyno" which, the manufacturer states, features longer life and reduced maintenance, and produces a smooth flow without pulsation. The engine and the pump on the AC-40A have

been moved to the rear of the sealer for easier operation, particularly in the sealing of transverse control joints.

The thermostat, propane pressure regulator, and other sensitive instruments, are grouped in a steel enclosure on the side of the sealer, where they are readily available for adjustment, yet protected against accidents and changes of setting. A simplification of piping and distribution systems has eliminated the need for extra valves and minimized the possibility of the material "freezing" in the pipes. An 18-hp air-cooled Wisconsin engine is used on the AC-40A. The engine is propane fired, and is equipped with a clutch and reduction gear.—Clipper Manufacturing Co., 2800 Warwick, Kansas City 8, Mo.

Horizontal-grate cooler

Designed to cool materials discharged from rotary kilns and sintering machines, a horizontal-grate cooler eliminates the extra headroom needed to accommodate the inclined grate of conventional units. Reduced height means less impacting steel, refractories, and chain curtain are required, making a lighter and more compact unit according to the manufacturer.

As in an inclined-grate cooler, movable and stationary grates alternate. But with the grates on a horizontal plane, material is pushed by the movable grates up and over the noses of the stationary ones. Resistance of the material against this movement causes immediate horizontal spread across the cooler bed, thereby permitting use of wider and shorter coolers where needed.

The cooler features a control system in which the temperature of the air used for the combustion of fuel in the kiln or furnace determines the amount of air admitted under the grate compartment at the hot end of the cooler. Under-grate pressure, in turn, determines the speed of the grates. The rate of travel of the material bed is thus varied automatically to maintain a uniform thickness and, therefore, uniform resistance to the flow of cooling air. Hood kiln or furnace draft determines the amount of air vented to the atmosphere. Drive-shaft speed may be manually or automatically controlled to produce the desired depth of material on the cooler grates and the rate of movement of material through the cooler.—Fuller Co., Catasauqua, Pa.

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(Page Numbers refer to News Letter)

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Columbia-Southern Chemical Corporation 12

Forney's, Inc. 25

Intrusion-Prepakt, Inc. 2

Lone Star Cement Corporation ..... Inside Front Cover

Marathon 23

Servicised Products Corporation 30

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Techkote Company 32

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37-lb rotary drill

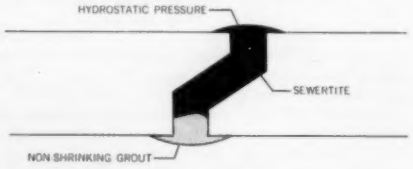
Winkie diamond drill tips the scales at only 37 lb, has a depth capacity up to 200 ft, and cuts up to 6 in. in diameter. The unit reportedly consumes about 2 gal. fuel per shift and as little as 1½ gal. of water per ft of drilling. An overhead drive principle allows up to 10-ft strokes without rechucking the drill rod. The manufacturer states that maximum core recovery and diamond bit economy are results of the machine's exacting dynamic balance, and that penetration speeds in excess of 1 ft per min have been obtained at depths of 100 ft.

Engineering features include a water-cooled gear box, safety clutch, built-in water swivel, and a convenient bulls-eye level. An aluminum-magnesium alloy construction is used in the drill and engine, which is assembled by units for service convenience. A 5¼-hp, 2-cycle, gasoline engine with a vacuum carburetion system allows continued all-angle drilling.—Wink Corp., 1518 North 117th St., Milwaukee 13, Wis.

Cold plastic sewer joint compound

A method of employing natural hydrostatic pressure as an aid in sealing concrete sewer pipe joints has been devised with the use of a cold plastic sewer joint compound called Sewertite by Edward Witter of Witter-Gaddis, Inc., Schereville, Ind., contractors.

In a sewer project in Gary, Ind., involving some 4700 ft of precast, reinforced concrete pipe, one problem encountered was hydrostatic pressure at the bottom of the large sewer pipes running as high as 7 or 8 lb. Mr. Witter developed a method of using this pressure to advantage. The cold plastic sewer joint compound was applied with a trowel to the tongue end of the pipe to a thickness of about ¾ in. The pipe was lowered



into the trench, fitted in place, and the joint was then grouted on the inside with nonshrinking mortar. Hydrostatic pressure forces the joint compound against the inner grout, into minute openings, effectively sealing the joints against water infiltration according to Mr. Witter.

The manufacturer claims that the plastic compound will not crack with ground movement, that the pressure on this plastic compound actually increases its sealing power.—Philip Carey Mfg. Co., Wayne at Cooper, Cincinnati, Ohio

Detailing Practices

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The ***Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)*** incorporates under one cover the previous separate editions on detailing of buildings and highway structures. It is a correlation of the latest improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. Sections on detailing and fabricating shop practice are translated into practical examples in typical drawings. Spiral bound to lie flat, it is the only publication of its kind in English and is meeting wide acclaim among designers, draftsmen, and engineering schools.

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DISCUSSION, PROCEEDINGS V. 54

Discussion of papers published in the July through September, 1957, JOURNALS appears in the concluding pages of this March issue, as the Institute inaugurates its plan for quarterly publication of discussion. Papers published October through December, 1957, will be discussed in the June 1958 issue; discussion of papers published January through March, 1958, will be published in Part 2, September 1958 ACI JOURNAL. April through June discussion will appear in Part 2, December 1958 along with index and errata for V. 54.

Listed below are the papers whose discussion is published in this JOURNAL issue.

Disc. 54-1	**Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)**, ACI Committee 214	775
Disc. 54-2	**Under-Reinforced Concrete Beams Under Long-Term Loads**, Herbert A. Sawyer, Jr. and Jack E. Stephens	779
Disc. 54-3	**General Method for Analysis of Flat Slabs and Plates**, John F. Brotchie	787
Disc. 54-4	**Effect of a Waterproof Coating on Concrete Durability**, Warrington G. Mitchell	797
Disc. 54-7	**Design of Concrete Floors on Ground for Warehouse Loadings**, Paul F. Rice	799
Disc. 54-10	**Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete**, Gene M. Nordby and William J. Venuti	803
Disc. 54-11	**Review of Changes in the ACI Building Code Requirements for Reinforced Concrete**, Frank Kerekes	809
Disc. 54-14	**Destructive Impulse Loading of Reinforced Concrete Beams**, F. T. Mavis and M. J. Greaves	811